TECHICAL MEMORANDUM
CONCEPTUAL GEOTECHNICAL ASSESSMENT
FOR SEDIMENT REMOVAL
ASHLAND/NORTHERN STATES POWER LAKEFRONT SITE
ASHLAND, WISCONSIN

Prepared for:

UNITED STATES ENVIRONMENTAL PROTECTION AGENCY
Region V Emergency Response Branch
77 W. Jackson Boulevard
Chicago, Illinois 60604

Prepared by:

WESTON SOLUTIONS, INC.
Region V Superfund Technical Assessment and Response Team
750 East Bunker Court, Suite 500
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November 20, 2009

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The Ashland/Northern States Power Lakefront site is located on the shore of Chequamegon Bay, Lake Superior, in northern Wisconsin. The site includes approximately ten acres of contaminated lake-bottom sediment located immediately offshore. Weston Solutions, Inc. (WESTON®) was tasked with completing a preliminary geotechnical assessment of potential design and construction issues associated with removal of these near-shore sediments using dry excavation techniques. This technical memorandum describes the analyses completed as part of this assessment, assumptions included in the assessment, limitations of the analyses, and recommendations for additional evaluations.

Summary of Project Understanding

Contaminated sediments, which have been covered by a layer of wood chips and wood debris, have accumulated on the bay bed. The most heavily contaminated materials have been found to be located within 200 feet of the shoreline with less heavily contaminated materials located at a distance of up to approximately 800 feet from the shoreline. According to the Proposed Plan for the site, the preferred Remedial Option for sediment cleanup in the Chequamegon Bay of Lake Superior is to remove the more heavily contaminated near-shore materials using dry excavation methods, and removal of the less contaminated sediments located further from shore using wet excavation (i.e., mechanical dredging) methods.

An analysis of excavation bottom heave prepared by Foth Infrastructure & Environmental, LLC (June 1, 2009) suggested that basal heave, due to a significant artesian head in a deeper aquifer stratum, could pose a significant risk to worker safety, which could in turn negatively impact removal of the contaminated sediments using dry excavation techniques. Foth, in their memorandum, did acknowledge that they used a simplistic approach to evaluate basal heave. That is, they simply compared the total downward vertical stress at the top of the aquifer to the hydraulic uplift pressure created by the artesian head at the top of the aquifer to define a Factor of Safety (FS) against basal heave. This is noted to be both simplistic and conservative since it neglects the shear strength of the uplifted soils along the vertical sidewalls of the failure mass.

Objective

The objective of this assessment is to complete a more rigorous and thorough evaluation of not only basal heave but other failure mechanisms that could pose a potential risk to workers, the environment, and to the successful completion of the project. Recommendations will be provided with respect to the likelihood of successfully completing removal of contaminated near shore sediments using dry excavation techniques.

Design Considerations

A number of design considerations related to the ability to safely and successfully complete removal of near-shore sediments using dry excavation techniques have been identified as outlined below.

1. Structural stability of the sheet pile retaining wall required to dry excavate bay bottom sediments;
2. Upheaval of the bay bottom dry excavation surface using Foth's approach modified to include the shear strength along the assumed vertical sidewalls of the failure mass;
3. Excavation bottom blowout due to shear failure of the cohesive aquitard soils induced by the dry excavation and the artesian head in the aquifer, which underlies the aquitard; and
4. Piping (i.e., liquefaction) of cohesionless bay bottom sand and silty sand sediments at the surface of the dry excavation due to upward hydraulic exit gradients.

Of these design considerations, structural stability of the sheet pile wall, excavation bottom blowout and piping of bay bottom sandy sediments are significant worker/equipment safety concerns and represent potential "fatal flaw" failure mechanisms. In contrast, upheaval of the bay bottom dry excavation surface simply indicates the potential for the bay bottom excavation surface to rise in elevation as a result of the upward artesian pressure in the underlying aquifer. This mechanism becomes more likely as the overburden load due to free water and excavated bay bottom sediments is sequentially removed in the dry excavation scenario. The "elastic extension of the soil below the excavation bottom due to load relief by excavating ..." is "only a problem of usability[1]." It is further noted that upheaval is a primary concern when it is necessary to achieve a specified design excavation bottom elevation for the construction of structural elements, such as floor slabs or footings. Upheaval on such projects may require the over-excavation of excavation bottom soils to achieve the targeted design excavation bottom elevation. The design elevation would depend on the estimated magnitude of upheaval of the excavation bottom. In the context of this analysis upheaval (basal heave) does not equate to failure.

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[1] *Geotechnical Engineering Handbook (Volume 3),* edited by Ulrich Smoltczyk, 2002, John Wiley & Sons.

Analyses

The following sections present summaries of the preliminary design analyses.

Introduction

The following analyses are based on the initial assumption that a series of sheet pile walls will be installed around the complete perimeter of the proposed dry excavation area at on-land and in-water locations consistent with a preliminary remedial action plan developed by others. This alignment is shown in Figure 1. The distance from the on-land shoreline to the in-water parallel sheet pile walls is understood to be about 200 feet. Following complete installation of the on-land and in-water sheet piling, the dry excavation footprint would reportedly be fully dewatered followed by dry excavation of contaminated bay bottom sediments within this footprint to a 5-foot depth.

Development of Engineering Properties and Subsurface Cross-Section

Preliminary analyses have been completed in support of each of the design considerations discussed above using available site subsurface exploration data (i.e., on-land and in-water test boring and well/piezometer logs) as well as limited geotechnical laboratory test data, which were available for this study. The development of a "Conceptual Design Subsurface Profile" and the physical/engineering properties of the various soil strata, which comprise this profile, is documented in Appendix A and summarized on Page 42 of this appendix. The selected physical and engineering properties of the four soil strata, which comprise this profile, are summarized in Table 1.

Table 1
Summary of Physical and Engineering Properties of Site Soils*

Soil Type	USCS Description	Total Unit Weight (pcf)	Undrained Shear Strength (psf)	Internal Friction Angle (deg)	Vertical Permeability (cm/sec)
Bay Bottom Granular Soils (Silty Sand and Sand)	SP/SM	101	0	26	$1x10^{-2}$
Aquifer (Sands and Silty/Clayey Sands)	SP/SM/SC	113	31	0	$1x10^{-3}$
Aquitard (Low Plasticity Clay Soil)	CL	124.5	660	0	$1x10^{-7}$
Aquitard (Low Plasticity Clayey Silt Soils)	ML	130.5	1250	0	$1x10^{-5}$

*Data from Appendix A, p.42
USCS- Unified soil classification system
pcf- pounds per cubic foot
psf- pounds per square foot
deg- degrees
cm/sec- centimeters per second

Information included on logs from water-based test borings 2900N/1500E and 2900N/2000E (i.e., physical descriptions and measured standard Penetration Resistance values [i.e., "N-values"]) were primarily used in the development of the Conceptual Design Subsurface Profile. Where necessary, these data were supplemented by on-land shoreline test borings/monitoring wells MW-24A, MW-25A and MW-26/26A. For example, the water-based test borings 2900N/1500E and 2900N/2000E did not fully penetrate the combined CL and ML strata, which comprise the aquitard. Therefore, the on-land shoreline borings/well logs were used to determine the aquitard thickness. Therefore, in using the three on-land test borings, the average of the combined ML and CL strata thicknesses shown on logs MW-24A (i.e., 32.5') and MW-25A (i.e., 23') was used to determine an average aquitard stratum thickness of 28' for use in the various stability analyses. (The thickness of the aquitard shown on MW-26/26A of 41 feet was conservatively neglected in the development of the average thickness.) Further information based on the Conceptual Design Subsurface Profile is summarized in Table 2.

Table 2
Preliminary Design Subsurface Cross-Section

Stratum	USCS Description	Surface Elevation (ft)	Thickness (ft)
Free Water	---	602	7.5
Bay Bottom Sands /Silty Sands	SP/SM	594.5	8.5
Aquitard	CL	586	8
Aquitard	ML	578	20
Aquifer	SC/SM/SP	558	To depth

USCS- Unified soil classification system
ft- feet

Sheet Pile Design

Five feet of contaminated sediment are to be removed from the bay bottom using dry excavation techniques. Based on the Conceptual Design Subsurface Profile, this will require the sheet pile wall to safely retain 7.5 feet of free water and 5 feet of sandy soil (bay bottom sediments). A cantilevered sheet piling installation is appropriate for this work. A conceptual sheet pile design was therefore completed using the developed subsurface profile modified for the above assumptions as documented on Page 4 of Appendix B. The analysis was completed using PROSHEET, a steel sheet piling design program developed by Skyline Steel Corporation. The results of the PROSHEET analysis are presented in Appendix B. The analysis indicated that the sheet geometry and structural properties presented in Table 3 would be required.

Table 3
Required Sheet Pile Properties[4]
(Conceptual Design)

Property	Value
Total length[1,2]	45 ft
Stickup length[3] (above excavation bottom)	15.5 ft
Embedment depth[3] (below excavation bottom)	27.4 ft
Minimum section modulus	25.19 in^3/ft
Steel grade/yield strength	A572 Grade 50

[1]The actual calculated length was 42.9 feet.
[2]The stickup length includes 3 feet of freeboard above elevation 602 feet, the bay water surface elevation.
[3]Includes a factor of safety (FS) of 1.3 on calculated embedment depth.
[4]Data from Appendix B
ft- feet
in^3/ft- cubic inches per foot

It should be noted that no external forces, such as those due to wave and ice loading, were included in the preliminary design.

Possible sheet pile sections, which satisfy the preliminary design criteria, include AZ-14/770 (25.2in^3/ft), PZC-14 (26.0in^3/ft) and PZ-27 (27.0in^3/ft). Due to the dewatering of the inside area of the sheet pile creating a differential in water pressure head alonf the sheet pile wall, the selected sheet pile section must have hot-rolled (i.e., watertight "ball and socket" geometry) interlocks to minimize water leakage into the dry excavation area.

Upheaval of the Bay Bottom Excavation Surface

Upheaval of the surface of the bay bottom excavation can be a concern if a saturated permeable soil strata, which tends to "confine" and therefore pressurize the pore water in the permeable layer to values greater than hydrostatic pressure, exists below an impermeable soil strata. In this instance, the impermeable stratum, as well as any overlying strata, have a tendency to be uplifted by the water pressure within the permeable layer. The significant artesian conditions in the aquifer at this site exacerbate this concern. The method used to determine upheaval of the lake bottom surface was similar to the method used in Foth's basal heave analysis with the exception that soil shear strength along the assumed vertical shear planes of the uplifted failure mass was included in the analysis. The modified upheaval analysis is presented in Appendix C. The analysis evaluated various failure mass geometries ranging from 1-foot by 1-foot up to 200 feet by 200 feet in plan dimensions as summarized in the table presented on page 12 of Appendix C. It is noted that the calculated Factor of Safety against upheaval decreased as the footprint size (area) of the failure mass increased. Assuming that a minimum FS value of 1.25 is appropriate for this analysis, the table in Appendix C indicates that the dry excavation plan footprint should not exceed 150 feet by 200 feet. This, in turn, suggests that it would be possible to install an in-water sheet pile wall approximately 200 feet from the shoreline as presently conceptualized as long as sheet pile walls perpendicular to this wall separated by no more than 150 feet were also installed to subdivide the dry excavation footprint into 150 feet by 200 feet cells before dewatering of any given cell to complete the dry excavation is permitted, see Figure 2.

Excavation Bottom Blowout

Excavation bottom blowout analyses are only applicable to saturated clayey soils under undrained conditions. This condition represents a potential rotational shear strength failure in the clayey soils which underlie an excavation bottom, and can be evaluated using several available quantitative procedures. These include the bearing capacity method, the slip circle method and the negative bearing capacity method. As discussed in detail in Appendix D, the negative bearing capacity procedure was selected for this study because it is widely used to evaluate excavation bottom stability, and typically yields a lower, and therefore, more conservative FS value as compared to the remaining two procedures.

In particular, a negative bearing capacity procedure developed by Bjerrum and Eide[2], which permits the FS value to be calculated for the case of 2 cohesive soil layers underlying the excavation bottom, was used for this stability evaluation. As documented in Appendix D, the analysis yielded a Factor of Safety value of 1.63 which exceeds the minimum recommended for this stability evaluation.

Exit Gradient Analysis

Vertically upward hydraulic exit gradients within a dry excavation cell footprint could potentially destabilize the granular excavation bottom soils (i.e., the SP/SM cohesionless materials), if sufficiently high. The nature of this process would be a fluidization of these materials similar to liquefaction, and represents a significant safety concern regarding workers and equipment which mobilize on the dry excavation surface. This phenomenon is referred to as "piping" in the geotechnical literature.

Two physical mechanisms, one natural and the other man-induced, could, on their own or in combination, result in piping in the excavation bottom SP/SM surficial soils. These include:

1. The upward gradient, which is naturally present in the SP/SM, CL and ML soil layers beneath the excavation bottom, induced by the artesian head in the underlying granular soil aquifer.

[2] Attachment to Appendix D, C.Y. Ou, Deep Excavation/theory and Practice, pp. 134-146

2. The upward gradient, which will be induced immediately adjacent of the sheet pile wall following its installation and dewatering of the free water inside a given 150-feet by 200-feet sheet piling cell, induced by the higher free water head in the bay water (elevation 602' surface) outside of the cell.

The first mechanism was evaluated and quantified using SEEP/W, a commercially available finite element software. The SEEP/W results are presented in Appendix E. Page 7 of the appendix depicts the upward flow induced by the above mechanisms. In addition, page 9 of this appendix presents a plot of the estimated exit gradient relevant to the assumed dewatered dry excavation groundwater level (elevation 587.5') across the width of the excavation. The numerically insignificant average value of this exit gradient (i.e., 1.7×10^{-5} ft/ft) is assumedly due to the very significant head loss/water pressure loss induced in the upward flow as it migrates through the 28 feet aquitard thickness. Note also that the highest exit gradient predicted by SEEP/W (i.e., 2×10^{-5} ft/ft) occurs first inside the sheet piling retaining walls. This behavior is well known in geotechnical engineering practice.

In this regard, a second quantitative procedure was also completed to estimate this maximum upward exit gradient immediately adjacent to, and inside of the sheet pile walls. This analysis is also included in Appendix E, and is consistent with the very conservative assumption that the entire subsurface environment consists of the SP/SM granular soils which comprise the excavation bottom (i.e., the ML and CL aquiclude soils are assumed to be non-existent), and therefore, much lower head loss/water pressure loss occurs in the underflow beneath the sheet piling before the underflow emerges just inside the sheet pile walls at the maximum exit gradient. The calculated maximum exit gradient using this procedure was 0.187 ft/ft.

The above calculated exit gradient values based on the two physical mechanisms which create vertically upward flow toward the excavation bottom can be summed to define the exit gradient which, if significantly high, can induce piping in the SP/SM excavation bottom soils. This value (i.e., $0.187 + 1.7 \times 10^{-5} \approx 0.187$) can be compared to the critical gradient in the SP/SM soils to define a FS against piping instability. This latter value was calculated to be 1.034 based on laboratory test data (see Appendix E). The corresponding FS value was determined to be 5.53 (i.e., 1.034/0.187). This value is greater that the recommended minimum FS value of 4 to 5 for piping instability analyses.

Conclusions and Recommendations

It must initially be stated and understood by all relevant project participants that the analyses, conclusions and recommendations presented herein are based on limited site-specific geotechnical field and laboratory data, and therefore, must be considered preliminary and conceptual-level only. However, based on our work effort in completing this study, WESTON is of the opinion that the near-shore, bay bottom sediments likely can be safely removed using dry excavation techniques assuming that conceptual planning, final design engineering and implementation of the construction work are all properly executed.

It is also noted that, in order for a final design of the dry excavation alternative to be properly completed, additional geotechnical data are required. In particular, the thickness, shear strength and permeability of the CL and ML strata, which comprise the aquitard are of primary interest. In this regard, a geotechnical investigation program should be structured that will allow for the collection of the required data and should include both field investigations (e.g., test borings, cone penetrometer testing, *in-situ* vane shear testing) and laboratory testing (e.g., undrained triaxial shear strength, vertical permeability, unit weight, moisture content, and physical properties) on recovered split-spoon and Shelby tube samples of the encountered soils. The field investigation program should be structured to obtain data from across the entire footprint of the proposed dry excavation area. In this regard, variability in the subsurface data is likely, which may result in several different sheet piling designs along the alignment of the wall including, for example, the selection of different sheet piling sections and/or lengths for different portions of the alignment, and variation in the width (i.e., "B" dimension) of the sheet pile cells across the dry excavation footprint.

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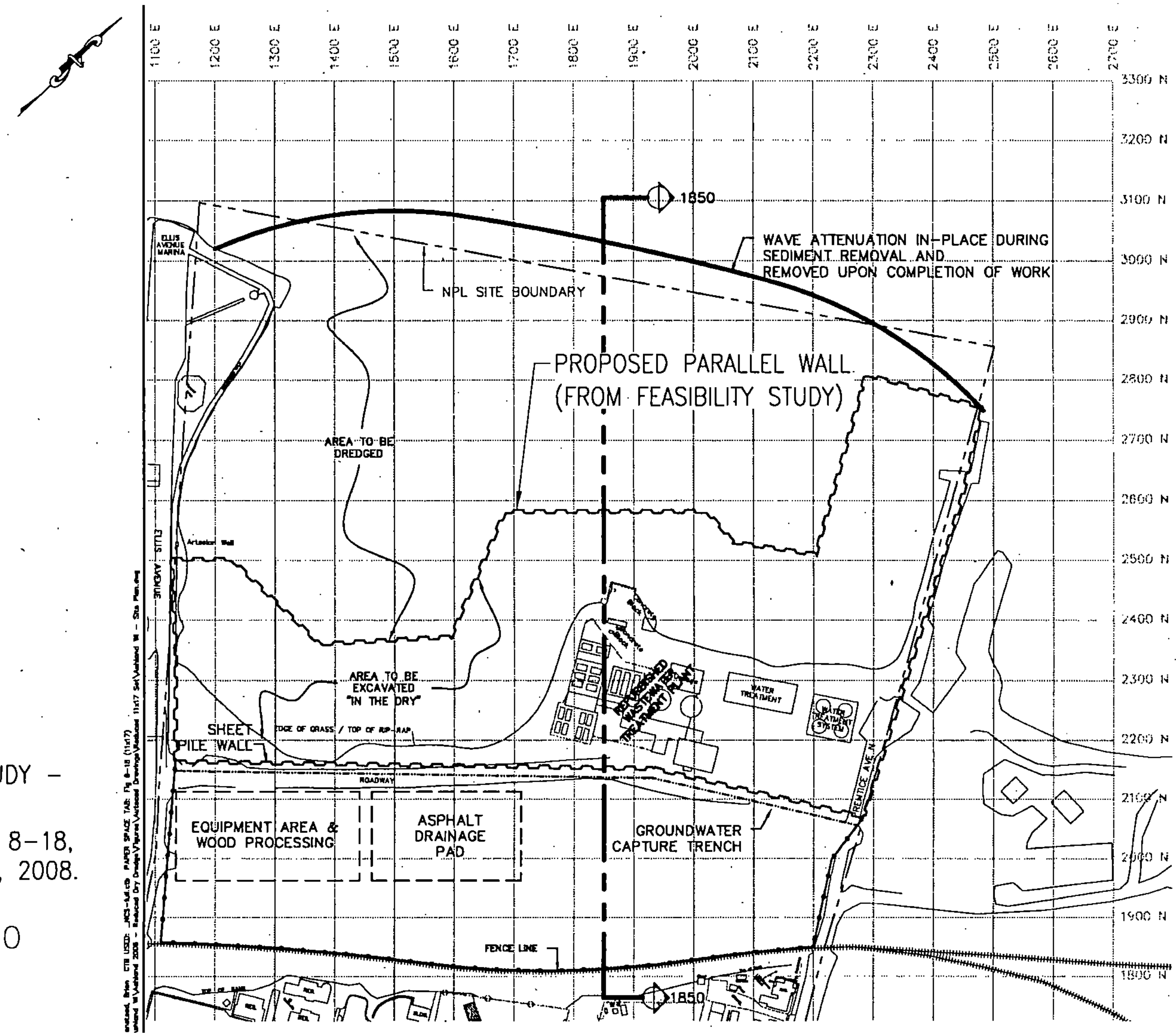

BACKGROUND IMAGE SOURCE:
"FINAL DRAFT REPORT FEASIBILITY STUDY – ASHLAND/NORTHERN STATES POWER LAKEFRONT SUPERFUND SITE" FIGURE 8-18, BY URS CORP., DATED NOVEMBER 21, 2008.

300 0 300

SCALE: 1"=300'-0"

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USEPA
ASHLAND/NSP LAKEFRONT SITE
ASHLAND, WI

FIGURE 1
PROPOSED PARALLEL SHEETPILING WALL

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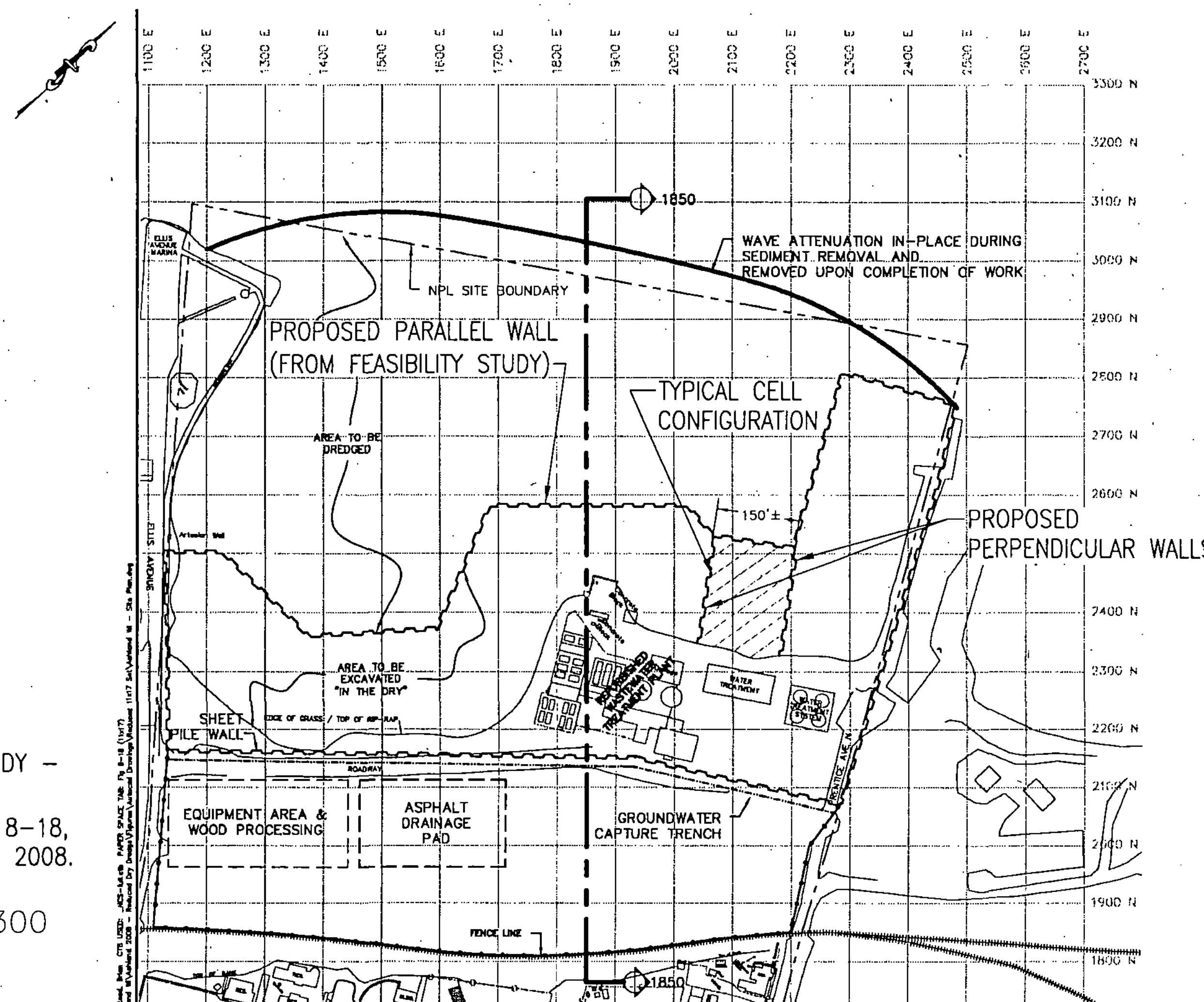

BACKGROUND IMAGE SOURCE:
"FINAL DRAFT REPORT FEASIBILITY STUDY – ASHLAND/NORTHERN STATES POWER LAKEFRONT SUPERFUND SITE" FIGURE 8-18, BY URS CORP., DATED NOVEMBER 21, 2008.

300 0 300

SCALE: 1"=300'-0"

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ASHLAND/NSP LAKEFRONT SITE
ASHLAND, WI

FIGURE 2
CONCEPTUAL SHEET PILING CELL CONFIGURATION

APPENDIX A

DEVELOPMENT OF
"CONCEPTUAL DESIGN SUBSURFACE PROFILE"

CLIENT/SUBJECT Ashland, Wis Superfund Site SHEET 1 of
W.O. NO.
TASK DESCRIPTION TASK NO.
PREPARED BY WLD DEPT DATE
MATH CHECK BY DEPT DATE
METHOD REV. BY DEPT DATE
APPROVED BY
DEPT DATE

Development of Conceptual Design Subsurface Profile For Completion Of Targeted Stability Analyses

A portion of the footprint of Chequamegon Bay in northern Wisconsin was determined to contain contamination in the bay sediments. The footprint of this contamination is shown in Figure 1 on page 2. The currently proposed remedial action consists of "dry excavation" of these sediments to a 5' depth between the shore of the bay and a 200'± offset from the shore. The remainder of the contaminated sediments footprint will consist of "wet excavation" of

(2)

LEGEND

- SEDIMENT SAMPLING LOCATION
- SOIL BORING LOCATION
- APPROXIMATE HORIZONTAL EXTENT OF OBSERVABLE CONTAMINATION
- 1994 SEH TEST PIT APPROXIMATE LOCATION AND NUMBER
- 1994 SEH TEMPORARY GROUNDWATER MONITORING WELL LOCATION AND NUMBER
- B-1 1994 SEH SOIL BORING LOCATION AND NUMBER
- EXISTING GROUNDWATER MONITORING WELL LOCATION AND NUMBER (NORTHERN ENVIRONMENTAL)
- AW-1 EXISTING ARTESIAN WELL LOCATION AND NUMBER (CITY OF ASHLAND)
- 605 TOPOGRAPHIC CONTOUR (1' INTERVAL, SITE DATUM)
- HYDRANT
- SANITARY SEWER MANHOLE
- STORM SEWER MANHOLE
- TELEPHONE PEDESTAL
- POWER POLE
- GUY ANCHOR
- LIGHT POLE
- POWER POLE WITH LIGHT
- UNDERGROUND GAS LINE
- UNDERGROUND ELECTRIC LINE
- UNDERGROUND TELEPHONE LINE
- CULVERT
- PINE TREE
- TREE
- SHRUB / HEDGE
- SWAMP

FIGURE 1

NOTES:
1. OBSERVABLE CONTAMINATION INCLUDES: VISUALLY APPARENT SHEENING, STAINING AND / OR NOTICEABLE PETROLEUM-LIKE ODOR.

NO.	DATE	ISSUE/REVISIONS	DRAWN BY	DESIGN	FIELD REVIEW	QC CHECK
1	07/12/96		JLE 05/94			SEH 7/96

SEH ENGINEERS • ARCHITECTS • PLANNERS

ASHLAND LAKEFRONT PROPERTY

FIGURE 3
EXTENT OF OBSERVABLE CONTAMINATION

PROJ. NO. WDNR9401 | DATE 07/12/96 | 3 / 8

F:\WASTE\WDNR\9401\REMEDIAL\FIG7

CLIENT/SUBJECT ____ W.O. NO. ____

TASK DESCRIPTION ____ TASK NO. ____

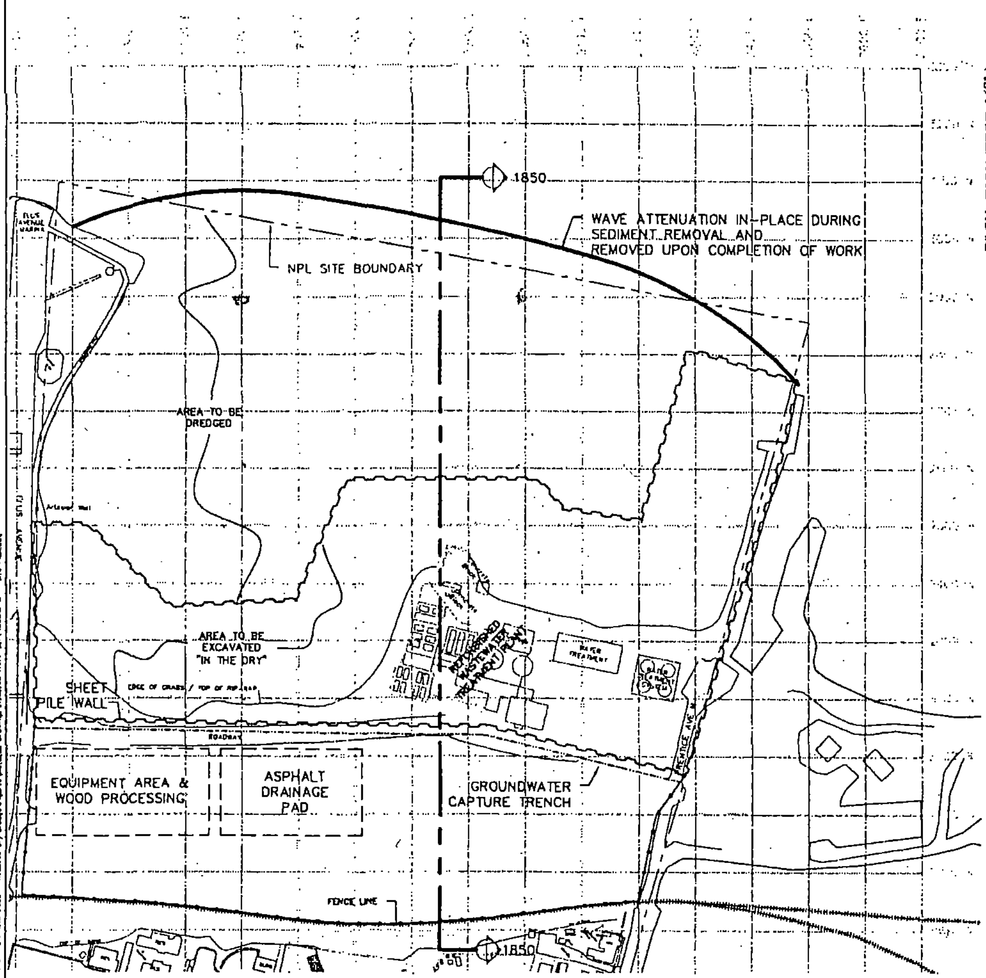
PREPARED BY ____ DEPT ____ DATE ____ APPROVED BY

MATH CHECK BY ____ DEPT ____ DATE ____

METHOD REV. BY ____ DEPT ____ DATE ____ DEPT ____ DATE ____

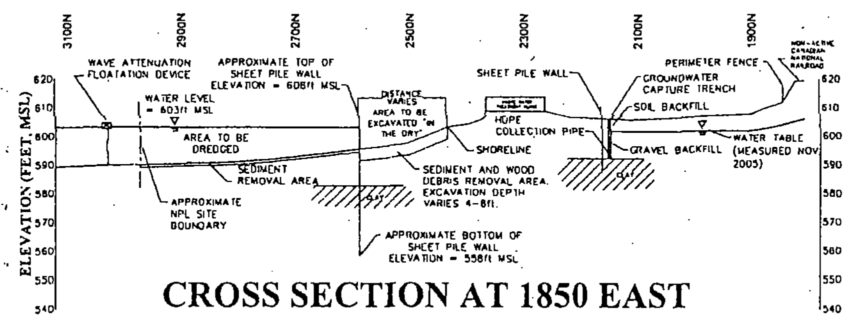
these materials using dredging techniques. The "dry excavation" area will be sheeted around its perimeter using steel sheet piling, completely dewatered, and the sediments excavated to the 5' depth using appropriate excavation and hauling equipment. The proposed alignment of the sheet pile wall is shown on Figure 2/page 4. Cross sections of the bay bottom geologic profile cut in a "site N/S" direction are shown in Figures 3 & 4 on pages 5 & 6.

Several stability scenarios are relevant to this study regarding the "dry excavation". These include



NOTES:

1) SECTIONS OF THE WAVE ATTENUATION DEVICE WOULD BE DESIGNED TO ALLOW BARGE ACCESS FOR INSTALLATION OF THE CONTAINMENT WALL LOCATED WITHIN THE BAY.



CROSS SECTION AT 1850 EAST

HORIZONTAL SCALE IN FEET

0 100 200 400

NORTH

SCALE IN FEET

0 100 200 400

LEGEND:

- = NPL SITE BOUNDARY
- = SHEET PILE WALL
- = GROUNDWATER CAPTURE TRENCH
- = FENCE LINE

FIGURE 2

REVISIONS		
NO.	NAME	DATE
#1	REDUCED DREDGE IN DRY BOUNDARY	08/18/08
#2	FIGURE NUMBER	10/21/08
#3	PAGE SIZE REDUCED	10/23/08

URS

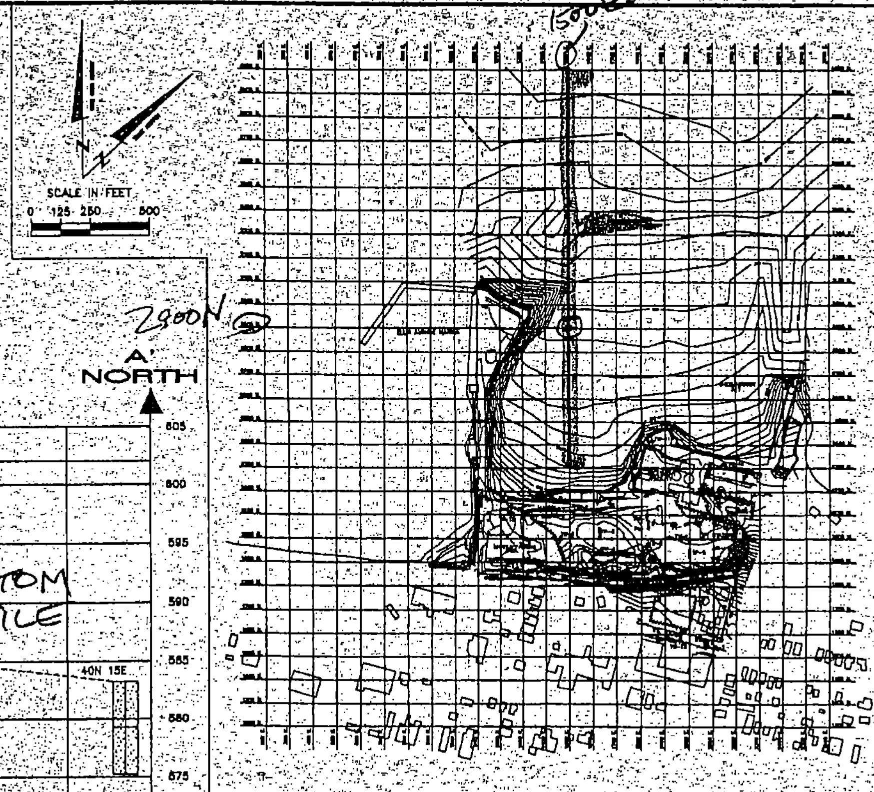
100 S. WACKER DRIVE
SUITE 500
CHICAGO, ILLINOIS 60606
(312) 939-1000 Tel
(312) 939-4198 Fax

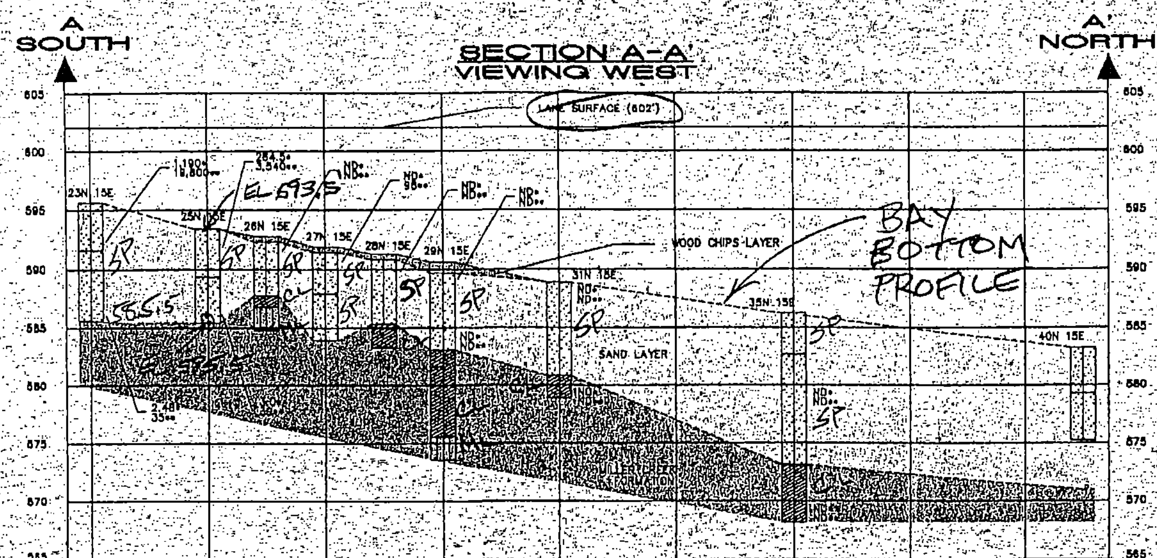
PROPOSED CONSTRUCTION ELEMENTS & CROSS SECTION
REDUCED DRY EXCAVATION ALTERNATIVE ASHLAND/NSP LAKEFRONT SITE ASHLAND, WISCONSIN

DRN BY. BKR	DATE: 10/21/08	PROJECT NO.	FIG. NO
CHK'D BY. JVH	DATE: 10/21/08	25688375.76000	8-1B

(4)

(5)





VERTICAL SCALE (FEET)
0 2.5 5 10

HORIZONTAL SCALE (FEET)
0 50 100 200

KEY (COLOR)

- WOOD CHIP LAYER
- SAND LAYER
- MILLER CREEK FORMATION

FIGURE 3

NOTES:

1. ELEVATIONS ARE SHOWN IN REFERENCE TO SITE DATUM.
2. FOR THE PURPOSE OF ILLUSTRATING SUBSURFACE CONDITIONS ON THE CROSS SECTIONS, SOME OF THE BORING LOGS HAVE BEEN SIMPLIFIED. FOR A DETAILED DESCRIPTION OF SUBSURFACE CONDITIONS AT INDIVIDUAL BORINGS, REFER TO SOIL BORING LOGS.
3. HORIZONTAL DISTANCES ARE MEASURED WITH RESPECT TO THE CENTER OF EACH SOIL BORING LOCATION.
4. * DENOTES CONCENTRATION OF TOTAL PAH (mg/kg).
5. ** DENOTES CONCENTRATION OF TOTAL PVOC (ug/kg).

LEGEND

- = WOOD CHIPS
- =CL LEAN CLAY
- =ML SILT
- =SM SILTY SAND
- =SP SAND
- WELL SCREEN INTERVAL

NO.	DATE	ISSUE/REVISIONS	DRAWN BY	DESIGN	FIELD REVIEW	QC CHECK
1	07/12/96		JLE 05/96			SEG 7-96

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ASHLAND LAKEFRONT PROPERTY

FIGURE 6
GELOGICAL CROSS SECTION A-A'

PROJ. NO. WDNR9401
DATE 07/12/96

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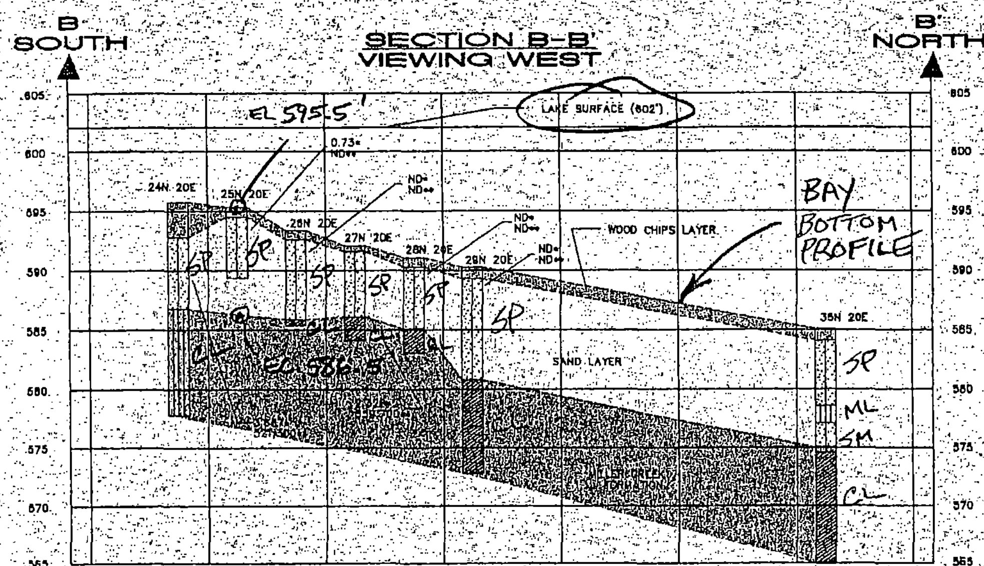
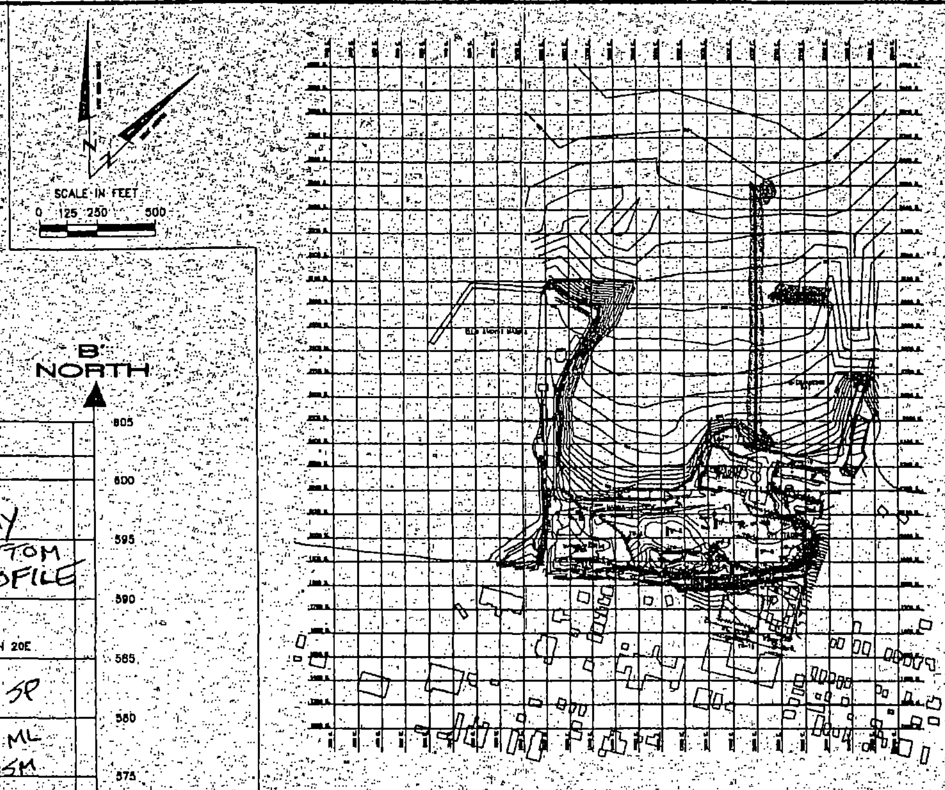
(6)

SECTION B-B'
VIEWING WEST





VERTICAL SCALE (FEET): 0, 2.5, 5, 10

HORIZONTAL SCALE (FEET): 0, 50, 100, 200

KEY (COLOR)

- WOOD CHIP LAYER
- SAND LAYER
- MILLER CREEK FORMATION

FIGURE 4

NOTES

1. ELEVATIONS ARE SHOWN IN REFERENCE TO SITE DATUM.
2. FOR THE PURPOSE OF ILLUSTRATING SUBSURFACE CONDITIONS ON THE CROSS SECTIONS, SOME OF THE BORING LOGS HAVE BEEN SIMPLIFIED. FOR A DETAILED DESCRIPTION OF SUBSURFACE CONDITIONS AT INDIVIDUAL BORINGS, REFER TO SOIL BORING LOGS.
3. HORIZONTAL DISTANCES ARE MEASURED WITH RESPECT TO THE CENTER OF EACH SOIL BORING LOCATION.
4. * DENOTES CONCENTRATION OF TOTAL PAH (mg/kg)
5. ** DENOTES CONCENTRATION OF TOTAL PVOC (ug/kg)

LEGEND

- = WOOD CHIPS
- =CL LEAN CLAY
- =ML SILT
- =SM SILTY SAND
- =SP SAND
- WELL SCREEN INTERVAL

NO.	DATE	ISSUE/REVISIONS	DRAWN BY	DESIGN	FIELD REVIEW	QC CHECK
1	07/12/96		JLE 05/96			XCC 7-96

SEH ENGINEERS ARCHITECTS PLANNERS

ASHLAND LAKEFRONT PROPERTY

FIGURE 7
GELOGICAL CROSS SECTION B-B'

PROJ. NO. WDNR9401 | DATE 07/12/96 | 7/8

F:\WASTE\DWG2\WASTE\WDNR\9401\REMEDIAL\FX03

CLIENT/SUBJECT ____ W.O. NO. ____

TASK DESCRIPTION ____ TASK NO. ____

PREPARED BY ____ DEPT ____ DATE ____

MATH CHECK BY ____ DEPT ____ DATE ____

METHOD REV. BY ____ DEPT ____ DATE ____

APPROVED BY ____ DEPT ____ DATE ____

the stability of the sheet piling retaining structure under the most critical fully dewatered / finished sediment excavation (i.e. 5' deep) scenario; and the stability of the dry finished excavation bottom against several possible instability potentials including basal heave, blow out due to a negative bearing capacity failure, and piping / liquefaction. These analyses are presented in other calculation packages. All of them were also modified to include the effects of hydrostatic uplift pressure from highly pressurized groundwater within the Copper Falls aquifer which is confined

WESTON SOLUTIONS

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by an overlying aquitard (Miller Creek Formation). The potentiometric head in the aquifer, which manifests itself as upward flowing water in wells which are screened near the aquitard/aquifer interface, which was used in these analyses was el 617.1' (Ref: Foth Infrastructure & Environment, 6/1/09 Rpt.).

Development of a representative Subsurface Cross Section for these analyses is discussed in the remainder of this package. For purposes of developing this profile, borings completed within the bay footprint were given priority, with borings completed along the shoreline supplementing this data where appropriate.

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METHOD REV. BY ____ DEPT ____ DATE ____

APPROVED BY ____ DEPT ____ DATE ____

1.) Bay Water Elev.:

- Based on Figures 3 + 4 (See pgs 5+6), the free water surface elevation of the bay is 602'.

2.) Bay Sediments Surface Elev.:

- As shown in Figure 2, page 4 the 2900N coordinate line is representative of the sheet pile alignment which will be positioned within the bay approx 200' from the shore.

- From Figures 3 + 4:

a) Elev bay sediments @ Boring 25N/15E = 593.5'

b.) Elev bay sediments @ Boring 25N/20E = 595.5

04P-0685

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MATH CHECK BY ____ DEPT ____ DATE ____

METHOD REV. BY ____ DEPT ____ DATE ____ DEPT ____ DATE ____

- Use ave value (= 594.5') as bay sediments surface elev. in cross section.

3.) Bay Water Depth:

- from above data, this value is el 602' - el 594.5' = 7.5'

4.) Surficial Bay Water Sediments

- Based on Figs 3 + 4, materials are granular & cohesionless and classify as "SP" (poorly graded sands) & SM (silty sand) according to the USCS.

- Also from Figs 3 & 4

a.) Bot. Elev. of SP/SM @ Boring 25N/15E = 585.5'

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b.) Bot. Elev of SP/SM @ Boring 25N/20E = 586.5'

- Therefore thickness (H) of SP/SM is:

a.) At 25N/15E = 593.5' - 585.5'
= 8'

b.) At 25N/20E = 595.5' - 586.5
= 9'

- use ave value (= 8.5') for $H_{SP/SM}$

- Select shear strength (c, ϕ) & total unit weight ($\gamma_T = \gamma_{SAT}$) for these materials.

a.) these values can all be correlated to a representative "N" value for this layer

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METHOD REV. BY DEPT DATE

APPROVED BY

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based on "Standard Penetration Testing" (ASTM D-1586). [N in "Blows/ft"

b.) Use boring logs available for this study which were "completed" in water" closest to the 2500N coordinate line in which "N" values were measured. These are Boring 29N/15E & 29N/20E. Logs for these 2 borings are presented on pgs 13 → 15.

c.) From 29N/15E Boring:

"N" (SP/SM) = 4, 5, ~~20~~, 8

From 29N/20E Boring:

"N" (SP/SM) = 5, 9, ~~18~~, ~~24~~, 4

- conservatively delete N = 24, 20 & 18 values & average remaining values.

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State of Wisconsin
Department of Natural Resources

Route To:
☐ Solid Waste ☐ Haz. Waste
☐ Emergency Response ☐ Underground Tanks
☐ Wastewater ☐ Water Resources
☐ Other

Soil Boring Log Information
Form 4400-122 7-91

Facility/Project Name	License/Permit/Monitoring Number	Boring Number
Ashland Lakefront Property		29N 15E

Boring Drilled By (Firm name and name of crew chief)	Date Drilling Started	Date Drilling Completed	Drilling Method
Maxim Technologies; Brad Davis	3/12/96	3/12/96	3.25" HSA

DNR Facility Well No.	WI Unique Well No.	Common Well Name	Final Static Water Level	Surface Elevation	Borehole Diameter
			Feet MSL	590.6 Feet	7.0 Inches

Boring Location
State Plane N, E
1/4 of 1/4 of Section T N,R
Lat 46° 35' 49"
Long 90° 52' 59"

Local Grid Location (If applicable)
2900 Feet ☒ N ☐ S 1500 Feet ☒ E ☐ W

County	DNR County Code	Civil Town/City/ or Village
Ashland	02	Ashland

Sample Number	Length (in) Recovered	Blow Counts	Depth In Feet	Soil/Rock Description And Geologic Origin For Each Major Unit	USCS	Graphic Log	Well Diagram	PID/FID	Standard Penetration	Moisture Content	Liquid Limit	Plastic Limit	P 200	RQD/ Comments
1	8	1-1-3-3 N=4	1.5	Wood chips				0.6/--	4					
				Brown, loose-medium dense SAND; little to some Silt, trace Gravel, no discernable odor SP/SM	SP SM									
2	11	1-1-4-2 N=5	3.0					4500/--	5					
3	12	6-9-11-11 N=20	4.5					0.2/--	20					
4	10	6-6-2-4 N=8	6.0	7.5'				-	8					
5	8	4-3-5-6 N=8	7.5 / 9.0	Brown, medium stiff, lean CLAY; some Sand CL 7.5'	CL			200/--	8					
6	16	3-4-6-7 N=10	10.5 / 12.0					140/--	10					
7	20	Shelby Tube	13.5	15'				-						
8	20	6-13-17-20 N=30	15.0 / 16.5	Brown, medium dense-dense SILT; some fine grained Sand ML	ML			80/--	30					
				End of Boring @ 17.0' → No aquifer penetration										

I hereby certify that the information on this form is true and correct to the best of my knowledge.

Signature: [signature]

Firm: SEH
SEH 421 Frenette Drive
Chippewa Falls, WI. 54729
Tel: 715-720-6200, Fax: 715-720-6300

This form is authorized by Chapters 144, 147 and 162, Wis. Stats. Completion of this report is mandatory. Penalties: Forfeit not less than $10 nor more than $5,000 for each violation. Fined not less than $10 or more than $100 or imprisoned not less than 30 days, or both for each violation. Each day of continued violation is a separate offense, pursuant to ss 144.99 and 162.06, Wis. Stats.

State of Wisconsin
Department of Natural Resources

Route To:
☐ Solid Waste ☐ Haz. Waste
☐ Emergency Response ☐ Underground Tanks
☐ Wastewater ☐ Water Resources
☐ Other

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Soil Boring Log Information

Form 4400-122 7-91

Facility/Project Name	License/Permit/Monitoring Number	Boring Number	
Ashland Lakefront Property		29N 20E	
Boring Drilled By (Firm name and name of crew chief)	Date Drilling Started	Date Drilling Completed	Drilling Method
Maxim Technologies; Brad Davis	3/11/96	3/11/96	3.25" HSA

DNR Facility Well No.	WI Unique Well No.	Common Well Name	Final Static Water Level	Surface Elevation	Borehole Diameter
			Feet MSL	590.3 Feet	7.0 Inches

Boring Location: State Plane N, E; 1/4 of 1/4 of Section T N,R; Lat 46° 35' 49"; Long 90° 52' 59"

Local Grid Location (If applicable): 2900 Feet ☒ N ☐ S; 2000 Feet ☒ E ☐ W

County	DNR County Code	Civil Town/City/ or Village
Ashland	02	Ashland

Sample Number	Length (in) Recovered	Blow Counts	Depth In Feet	Soil/Rock Description And Geologic Origin For Each Major Unit	U S C S	Graphic Log	Well Diagram	PID/FID	Standard Penetration	Moisture Content	Liquid Limit	Plastic Limit	P 200	RQD/ Comments
1	1	0-1-4-4 N=5		Wood chips				0.3/--	5					
			1.5	Brown, loose-medium dense, SILTY SAND; trace Gravel, no discernable odor	SM									
2	16	2-1-8-11 N=9	3.0					0.6/--	9					
3	19	6-6-12-12 N=18	4.5	SM				0.4/--	18					
4	19	6-12-12-6 N=24	6.0					0.5/--	24					
			7.5											
5	12	4-2-2-4 N=4	9.0	9.5'				0.2/--	4					
			10.5	Brown, medium stiff-stiff, lean, SANDY CLAY	CL									
6	12	3-6-6-9 N=12	12.0	CL 8'				0.6/--	12					
7	21	Shelby Tube	13.5					--						
			15.0											
8	24	2-3-3-3 N=6	16.5	17.5' EOB @ 17.5'				1.2/--	6					

I hereby certify that the information on this form is true and correct to the best of my knowledge.

Signature: John E. Guhl

Firm: SEH

SEH 421 Frenette Drive
Chippewa Falls, WI. 54729
Tel: 715-720-6200, Fax: 715-720-6300

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State of Wisconsin
Department of Natural Resources

Soil Boring Log Information Supplement
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Boring Number 29N 20E Use only as an attachment to Form 4400-122.

Sample: Number	Sample: Length (in) Recovered	Blow Counts	Depth In Feet	Soil/Rock Description And Geologic Origin For Each Major Unit	U S C S	Graphic Log	Well Diagram	PID/FID	Soil Properties: Standard Penetration	Moisture Content	Liquid Limit	Plastic Limit	P 200	RQD/ Comments
				End of Boring @ 17.5'	no aquifer penetration									

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$$N_{ave} = \frac{4+5+8+5+9+4}{6}$$

$$= 5.8 \rightarrow \text{use } N_{rep} = 6$$

d) then for the SP/SM materials

- $\gamma_T = 101$ pcf (see Table pg 17)

- $c = 0$ psf (granular soils)

- $\phi = \sqrt{20N} + 15^\circ$

$$= \sqrt{(20)(6)} + 15^\circ = 25.95^\circ$$

$\downarrow$ use 26°

Ref: Kishida, H.
"Ultimate Bearing Capacity of Piles Driven Into Loose Sand"; Soil and Foundations Vol 7, #3, pgs 20-29

CORRELATION BETWEEN N-VALUES AND UNIT WEIGHT

Fine Grained Soils

N (bpf)	γ_{total} (pcf)
0	95
1	106
2	110
3	114
4	117
5	118.5
6	120
7	122
8	123
9	124.5
10	125.5
11	126.5
12	127.5
13	128
14	129
15	129.5
16	130
17	130.5
18	131
19	131.5
20	132
21	132.25
22	132.5
23	133
24	133.5
25	134
26	135
27	135
28	135
29	135
30	135
31	135
32	135
33	135
34	135
35	135
36	135
37	135
38	135
39	135
40	135
41	135
42	135
43	135
44	135
45	135
46	135
47	135
48	135
49	135
50	135
51	135
52	135
53	135
54	135
55	135
56	135
57	135
58	135
59	135
60	135

Coarse Grained Soils

N (bpf)	γ_{total} (pcf)
0	80
1	85
2	90
3	93
4	96
5	98.5
6	101
7	103.5
8	106
9	108
10	110
11	111
12	112
13	113.25
14	114.5
15	115.25
16	116
17	116.75
18	117.5
19	118.25
20	119
21	119.75
22	120.5
23	120.75
24	121
25	122
26	123
27	123.5
28	124
29	124.5
30	125
31	125.5
32	126
33	126.5
34	127
35	127.5
36	128
37	128.5
38	129
39	129.5
40	130
41	130.5
42	131
43	131.5
44	132
45	132.5
46	133
47	133.5
48	134
49	134.5
50	135
51	135
52	135
53	135
54	135
55	135
56	135
57	135
58	135
59	135
60	135

WESTON SOLUTIONS

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5.) <u>Aquitard</u>

— Based on borings 29N/15E & 29N/20E, the low permeability aquitard consists of low plasticity clay soils (CL) underlain* by low plasticity clayey silt soils (ML). [It is noted that the CL stratum was encountered in both test borings, while only boring 29N/15E encountered the underlying ML soils approx 2' above the bottom of the boring. It is therefore assumed that boring 29N/20E was not deep enough to encounter the ML stratum.

* For shoreline boring MW-24A & MW-26/26A, the ML soils overlie the CL soils. For Boring MW-25A, only the CL soils were encountered. (See subsequent discussion)

A.) <u>CL Stratum</u>:

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a.) Based on logs of these borings (see pgs 13 & 14):

$H_{CL} = 7.5'$ (Boring 29N/15E)

$H_{CL} = 8'$ (Boring 29N/20E)

use $H_{CL}(ave) = 7.75' \rightarrow$ use $8'$

b.) "N" values in CL stratum:

$N = 8, 10$ (Boring 29N/15E)

$N = 12, 6$ (Boring 29N/20E)

use $N_{REP} = N_{ave} = \frac{8+10+12+6}{4} = 9$ BPF

c.) From pg 17 & "fine grained soil classification":

$\gamma_T = 124.5$ pcf

d.) Assume undrained shear strength behavior in these soils

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$\phi = \phi_u = 0°$

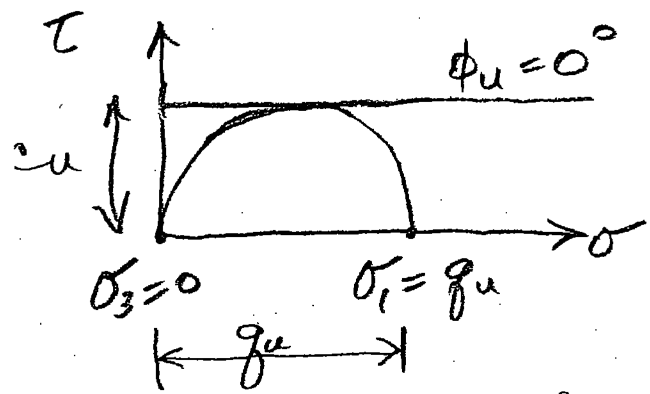
$c_u > 0$

- estimate from page 21 Figure

- Note that Figure yields q_u = unconfined compressive strength as a function of "N" & soil type/plasticity.

- For N = 9 & clays of low plasticity (CL) curve

$q_u = .66$ TSF



& $C_u = \frac{q_u}{2} = .33$ TSF $= 660$ psf

Note from pg 21 Table that this value correlates well to the ave. measured PP value of 7 PP readings in the CL soils

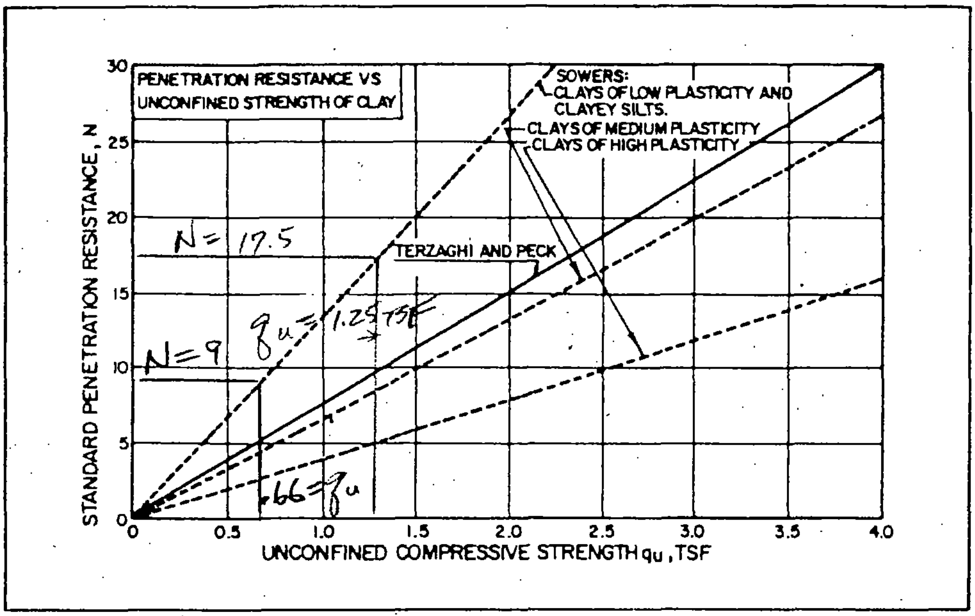


FIGURE 4
Correlations of Standard Penetration Resistance

Pocket Penetrometer (PP) Readings

Project: Ashland Lakefront
Project #: WIDNR9401
Date: 4/24/96

Sample #	Location	Depth (ft.)	Reading * (tons/sq. ft.)
S-284	2900N 1500E	13-15	0.60 = 600 psf
S-285	2900N 2000E	13-15	1.00 = 1000 psf
S-286	2100N 1000E	15.5-17.5	1.25 = 1250 psf
S-287	X 3500N 2000E	13-15	~~1.50~~
S-288	2700N 2500E	13-15	0.60 = 600 psf
S-294	2700N 2100E	6-8	0.50 = 500 psf
S-306	X 3100N 2300E	12-14	~~1.40~~
S-307	X 3100N 1500E	8-10	~~1.60~~
S-308	2700N 1700E	6-8	0.50 = 500 psf
S-310	2600N 1200E	6-8	0.50 = 500 psf

$$PP_{AVE} = \frac{\sum_{i=1}^{7} PP_i}{7} = 707 \text{ psf}$$

* Note: The PP measures the unconfined compressive strength of cohesive soils in tons/ft² (tsf)

X → Boring location too distant from proposed dry excavation footprint. Therefore, disregard PP rdg.

g:\wastl\pock

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APPROVED BY

DEPT DATE

B.) ML Stratum:

- Boring 29N/15E only penetrated 2' into this stratum. One measured N value (i.e. 30 blows/ft) was measured. Since this boring did not fully penetrate this stratum, its thickness is unknown from this boring.

- Boring 29N/20E did not penetrate this stratum at all, therefore, no data is available from this boring.

- Based on the above, data from land borings MW-24A, MW-25A & MW-26/26A will be used to develop

WESTON SOLUTIONS

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representative data for this stratum

Logs for these borings is presented on pages 24 → 34 & is summarized as follows:

I.) MW-24A

a.) Aquitard Thickness (CL & ML)

$H = 11' + 21.5' = 32.5'$

ML (12' → 23')

CL (23' → 44.5')

b.) Measured N values in ML

$N = 19, 16$

II.) MW-25A

a.) Aquitard Thickness (CL & ML)

$H = 22' + 1' = 23'$

CL (14' → 36')

Clay w/ sand (36' → 37')

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State of Wisconsin
Department of Natural Resources

SOIL BORING LOG INFORMATION

Form 4400-122 7-91

Route To:

- ☐ Solid Waste
- ☐ Wastewater
- ☐ Emergency Response
- ☐ Haz. Waste
- ☐ Underground Tanks
- ☐ Water Resources
- ☐ Other ______

Facility / Project Name	License/Permit/Monitoring Number	Boring Number
Ashland / NSP Lakefront Superfund Site		MW-24A

Boring Drilled By (Firm name and name of crew chief)	Date Drilling Started	Date Drilling Completed	Drilling Method
Boart Longyear - Paul Dickinson	05 / 12 / 04 (MM DD YY)	05 / 13 / 04 (MM DD YY)	4 1/4" ID HSA

DNR Facility Well No.	WI Unique Well No.	Common Well Name	Final Static Water Level	Surface Elevation	Borehole Diameter
		MW-24A	___ Feet MSL	___ Feet MSL	8.3 inches

Boring Location
State Plane ______ N. ______ E S/C/N | Lat ___ | Long ___
SW 1/4 of NW 1/4 of Section 33 T 48 N, R 4 E

Local Grid Location (If Applicable): ___ Feet ☐ N ☐ S ___ Feet ☐ E ☐ W

County	DNR County Code	Civil Town / City / or Village
Ashland	0 2	City of Ashland

Sample Number	Length Recovered (N)	Blow Counts (N)	Depth in Feet	Soil/Rock Description And Geologic Origin For Each Major Unit	USCS	Graphic Log	Well Diagram	PID/FID	Standard Penetration	Moisture Content	Liquid Limit	Plastic Limit	P 200	RQD/Comments
			0	Grassed area										
			0–6	FILL, CLAY, silty, trace gravel, moist, soft, low plasticity, dark brown. Contains brick fragments.										
1	16	1,1 1,2	6–11	FILL, wood ships, dark yellow brown, coal tar odor.					2					
2	12	3,3 4,5	11–12	SAND, fine grained, some silt, wet, loose, poorly graded, dark reddish brown.	SM				7					
			12–14	- Driller reports hard drilling at 12 feet. ML	ML									

I hereby certify that the information on this form is true and correct to the best of my knowledge.

Signature | Firm NewFields, Madison, Wisconsin

State of Wisconsin
Department of Natural Resources

SOIL BORING LOG INFORMATION SUPPLEMENT

Form 4400-122A 7-91

Boring Number MW-24A

Sample Number	Length Recovered (N)	Blow Counts (N)	Depth in Feet	Soil/Rock Description And Geologic Origin For Each Major Unit	USCS	Graphic Log	Well Diagram	PID/FID	Standard Penetration	Moisture Content	Liquid Limit	Plastic Limit	P 200	RQD/Comments
3	20	3,8 11,15 N=19	15–17	SILT, very moist, very stiff, non-plastic, dark reddish brown. ML	ML				19					
4	20	4,7 9,10 N=16	20–22						16					
5	22	5,8 13,14	25–27	CLAY, silty, very stiff, low plasticity, moist, dark reddish brown.	CL				21					
6	20	1,2 2,3	30–32	CLAY, silty, medium stiff, low plasticity, very moist, dark reddish brown.	CL -ML				4					
7	8		35–36											

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Department of Natural Resources

SOIL BORING LOG INFORMATION SUPPLEMENT
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Boring Number MW-24A

Sample Number	Length Recovered (N)	Blow Counts (N)	Depth in Feet	Soil/Rock Description And Geologic Origin For Each Major Unit	USCS	Graphic Log	Well Diagram	PID/FID	Standard Penetration	Moisture Content	Liquid Limit	Plastic Limit	P 200	RQD/Comments
7	8		37–40	CLAY, silty, medium stiff, low plasticity, very moist, dark reddish brown.	CL ML									
8	20	15,18 9,12	41–44	CLAY, silty, trace fine sand, trace fine gravel, moist, very stiff, low plasticity, dark reddish brown.					~~17~~ 27					
9	22 N=18	30,8 10,10	45–47	SAND, some clay, little gravel, wet, medium dense, poorly graded, dark reddish brown. SC	SC				18					
9	20 N=32	13,15 17,18	48–52	SP SAND, medium to coarse grained, trace silt, trace fine gravel, wet, dense, poorly graded, reddish brown.	SP				32					
			53–58	EOB @ 52 ft, set well MW-24A at 51 ft.										

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State of Wisconsin
Department of Natural Resources

Route To:

- ☐ Solid Waste
- ☐ Wastewater
- ☐ Emergency Response
- ☐ Haz. Waste
- ☐ Underground Tanks
- ☐ Water Resources
- ☐ Other ______

SOIL BORING LOG INFORMATION

Form 4400-122 7-91

Facility / Project Name	License/Permit/Monitoring Number	Boring Number
Ashland / NSP Lakefront Superfund Site		MW-25A

Boring Drilled By (Firm name and name of crew chief)	Date Drilling Started	Date Drilling Completed	Drilling Method
Boart Longyear - Paul Dickinson	05 / 17 / 04 (MM DD YY)	05 / 18 / 04 (MM DD YY)	4 1/4" ID HSA

DNR Facility Well No.	WI Unique Well No.	Common Well Name	Final Static Water Level	Surface Elevation	Borehole Diameter
		MW-25A	Feet MSL	Feet MSL	8.3 inches

Boring Location
State Plane ______ N. ______ E S/C/N
SW 1/4 of NW 1/4 of Section 33 T 48 N, R 4 E
Lat — — —
Long — — —
Local Grid Location (If Applicable)
______ Feet ☐ N ☐ S ______ Feet ☐ E ☐ W

County	DNR County Code	Civil Town / City / or Village
Ashland	0 2	City of Ashland

Sample Number	Sample Length Recovered (N)	Blow Counts (N)	Depth in Feet	Soil/Rock Description And Geologic Origin For Each Major Unit	USCS	Graphic Log	Well Diagram	PID/FID	Standard Penetration	Moisture Content	Liquid Limit	Plastic Limit	P 200	ROD/Comments
				Grassed area										
			1–14	No samples collected above 15 feet. See boring log for P-25 for soil descriptions.										

I hereby certify that the information on this form is true and correct to the best of my knowledge.

Signature	Firm NewFields, Madison, Wisconsin

This form is authorized by Chapters 144.147 and 162, Wis. Stats. Completion of this report is mandatory. Penalties: Forfeit not less than $10 nor more than $4,000 for each violation. Fines not less than $10 or more than $100 or imprisoned not less than 30 days, or both for each violation. Each day of continued violation is a separate offense, pursuant to ss 144.99 and 162.06, Wis. Stats

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SOIL BORING LOG INFORMATION SUPPLEMENT

Form 4400-122A 7-91

Boring Number MW-25A

Sample Number	Length Recovered (in)	Blow Counts (N)	Depth in Feet	Soil/Rock Description And Geologic Origin For Each Major Unit	USCS	Graphic Log	Well Diagram	PID/FID	Standard Penetration	Moisture Content	Liquid Limit	Plastic Limit	P 200	RQD/Comments
1	18	3,5 6,5	15–17	CLAY, silty, moist, low plasticity, stiff, reddish brown	CL				11					
2	20	5,11 19,13	20–22	CLAY, silty, very stiff, low plasticity, moist, dark reddish brown, trace sand and gravel CL					30					
3	20	8,11 12,14	25–27	CLAY, silty, very stiff to hard, low plasticity, moist, dark reddish brown.	CL				~~33~~ 23					
4	22	8,12 20,18	30–32	CLAY, silty, hard, low plasticity, moist, reddish brown.	CL				32					
5	20	9,11 16,21	35–36	CLAY, as above to 36 feet					27					

State of Wisconsin
Department of Natural Resources

SOIL BORING LOG INFORMATION SUPPLEMENT

Form 4400-122A 7-91

Boring Number MW-25A

Sample Number	Sample Length Recovered (N)	Blow Counts (N)	Depth in Feet	Soil/Rock Description And Geologic Origin For Each Major Unit	USCS	Graphic Log	Well Diagram	PID/FID	Standard Penetration	Moisture Content	Liquid Limit	Plastic Limit	P 200	RQD/Comments
			37	CLAY, silty, with SAND, trace gravel, wet, poorly graded, dense, dark reddish brown	SC			CL ?						
			38	Sand seam at 37 feet Driller reports soft drilling at 37 feet, water is rising in augers										
6	20	8,9 15,23	41	SAND, fine to medium grained, medium dense, poorly graded, dark reddish brown	SP				24					
			43	EOB at 43 feet, set well MW-25A at 42 ft.										

N = 24

(30)

State of Wisconsin
Department of Natural Resources

Route To:
☐ Solid Waste
☐ Wastewater
☐ Emergency Response
☐ Haz. Waste
☐ Underground Tanks
☐ Water Resources
☐ Other ____

SOIL BORING LOG INFORMATION

Form 4400-122 7-91

Facility / Project Name: Ashland / NSP Lakefront Superfund Site	License/Permit/Monitoring Number:	Boring Number: MW-26
Boring Drilled By (Firm name and name of crew chief): Boart Longyear - Paul Dickinson	Date Drilling Started: 05/18/04 (MM/DD/YY)	Date Drilling Completed: 05/18/04 (MM/DD/YY)
Drilling Method: 4 1/4" ID HSA		
DNR Facility Well No.:	WI Unique Well No.:	Common Well Name: MW-26
Final Static Water Level: ____ Feet MSL	Surface Elevation: ____ Feet MSL	Borehole Diameter: 8.3 inches
Boring Location: State Plane ____ N. ____ E S/C/N; SW 1/4 of NW 1/4 of Section 33 T 48 N, R 4 E	Lat ___ Long ___	Local Grid Location (If Applicable): ____ Feet ☐ N ☐ S ____ Feet ☐ E ☐ W
County: Ashland	DNR County Code: 0 2	Civil Town / City / or Village: City of Ashland

Sample Number	Length Recovered (N)	Blow Counts (N)	Depth in Feet	Soil/Rock Description And Geologic Origin For Each Major Unit	USCS	Graphic Log	Well Diagram	PID/FID	Standard Penetration	Moisture Content	Liquid Limit	Plastic Limit	P 200	RQD/Comments
			0	Grassed area										
			0–5	FILL, silty clay, dark brown -gravel/rock encountered	CL									
1	16	2,3 4,4	5–6	FILL, CLAY, moist, firm, low plasticity, dark reddish brown					7					
			6–10	SAND, some silt, trace gravel, wet, loose, poorly graded, dark brown, broken glass present	SM									
2	8	2,3 1,3	10–12	FILL, SAND, fine to medium grained, trace gravel, some silt, wet, loose, poorly graded, dark brown, strong coal tar/sheen					4					
3	16	2,3 4,4	12–14	SAND, fine to medium grained, wet, loose, poorly graded, dark reddish brown	SP				7					

I hereby certify that the information on this form is true and correct to the best of my knowledge.

Signature	Firm: NewFields, Madison, Wisconsin

(31)

State of Wisconsin
Department of Natural Resources

SOIL BORING LOG INFORMATION SUPPLEMENT
Form 4400-122A 7-91

Boring Number MW-26

Sample Number	Sample Length Recovered (N)	Blow Counts (N)	Depth in Feet	Soil/Rock Description And Geologic Origin For Each Major Unit	USCS	Graphic Log	Well Diagram	PID/FID	Soil Properties: Standard Penetration	Moisture Content	Liquid Limit	Plastic Limit	P 200	RQD/Comments
4 N=	16	4,7 11,12 =18	15–16	SILT, trace fine sand, non-plastic, very stiff, wet, dark reddish brown, slight odor ML	ML				18					
			16	EOB at 16 feet, set well MW-26 at 15 feet										

17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33 34 35 36

(32)

State of Wisconsin
Department of Natural Resources

Route To:

- ☐ Solid Waste
- ☐ Wastewater
- ☐ Emergency Response
- ☐ Haz. Waste
- ☐ Underground Tanks
- ☐ Water Resources
- ☐ Other ______

SOIL BORING LOG INFORMATION

Form 4400-122 7-91

Facility / Project Name	License/Permit/Monitoring Number	Boring Number
Ashland / NSP Lakefront Superfund Site		MW-26A

Boring Drilled By (Firm name and name of crew chief)	Date Drilling Started	Date Drilling Completed	Drilling Method
Boart Longyear - Paul Dickinson	05 / 18 / 04 (MM DD YY)	05 / 18 / 04 (MM DD YY)	4 1/4" ID HSA

DNR Facility Well No.	WI Unique Well No.	Common Well Name	Final Static Water Level	Surface Elevation	Borehole Diameter
		MW-26A	______ Feet MSL	______ Feet MSL	8.3 inches

Boring Location
State Plane ______ N. ______ E S/C/N
SW 1/4 of NW 1/4 of Section 33 T 48 N, R 4 E
Lat — — —
Long — — —

Local Grid Location (If Applicable)
______ Feet ☐ N ☐ S ______ Feet ☐ E ☐ W

County	DNR County Code	Civil Town / City / or Village
Ashland	0 2	City of Ashland

Sample Number	Length Recovered (N)	Blow Counts (N)	Depth in Feet	Soil/Rock Description And Geologic Origin For Each Major Unit	USCS	Graphic Log	Well Diagram	PID/FID	Standard Penetration	Moisture Content	Liquid Limit	Plastic Limit	P 200	RQD/Comments
				Grassed area										
			1	No samples collected above 20 feet. See boring log for MW-26 for soil descriptions.										
			2											
			3											
			4											
			5											
			6											
			7											
			8											
			9											
			10											
			11											
			12											
			13											
			14											

I hereby certify that the information on this form is true and correct to the best of my knowledge.

Signature	Firm NewFields, Madison, Wisconsin

State of Wisconsin
Department of Natural Resources

SOIL BORING LOG INFORMATION SUPPLEMENT

Form 4400-122A 7-91

Boring Number. MW-26A

ML?
(see pg 31 where ML enc. @ 14' bgs)

Sample Number	Length Recovered (in)	Blow Counts (N)	Depth in Feet	Soil/Rock Description And Geologic Origin For Each Major Unit	USCS	Graphic Log	Well Diagram	PID/FID	Standard Penetration	Moisture Content	Liquid Limit	Plastic Limit	P 200	RQD/Comments
1	8	15,20 25,32 N=45	20–23	SILT, trace fine sand, wet, hard, non-plastic, dark reddish brown ML	ML				45					
2	20	5,6 7,7	24–26	CLAY, silty, trace sand, wet, stiff, low plasticity, dark reddish brown	CL				13					
3	0	5,6 7,6	30–32	No recovery, pushing rock	CL				13					
4	18	7,8 12,15	35–36	CLAY, silty, trace sand, moist, very stiff, low plasticity, dark reddish brown					20					

State of Wisconsin
Department of Natural Resources

SOIL BORING LOG INFORMATION SUPPLEMENT

Form 4400-122A 7-91

Boring Number MW-26A

Sample Number	Length Recovered (N)	Blow Counts (N)	Depth in Feet	Soil/Rock Description And Geologic Origin For Each Major Unit	USCS	Graphic Log	Well Diagram	PID/FID	Standard Penetration	Moisture Content	Liquid Limit	Plastic Limit	P 200	RQD/Comments
			37 38 39											
5	16	11,12 19,24	40 41 42	CLAY, silty, trace sand, moist, very stiff, low plasticity, dark reddish brown -sand seams present (<1/4" thick) below 41 feet	CL				31					
			43 44											
7	16	9,12 14,21	45 46 47	CLAY, silty, little sand, trace gravel, moist, very stiff, low plasticity, dark reddish brown	CL ML				26					
			48 49											
8	18	11,17 14,23	50 51 52	CLAY, as above -4 inch piece of wood encountered at 51.5 feet	CL ML				31					
			53 54	Driller reports soft drilling at 54 feet										
9	10 N=35	6,12 23,27	55 56 57	SAND, fine to medium grained, some silt, trace gravel, wet, dense, poorly graded, dark reddish brown	SM				37					
			58	EOB at 61 feet, set well MW-26A at 60 ft.										

WESTON SOLUTIONS

SHEET 35 of

CLIENT/SUBJECT W.O. NO.

TASK DESCRIPTION TASK NO.

PREPARED BY DEPT DATE

MATH CHECK BY DEPT DATE

METHOD REV. BY DEPT DATE

APPROVED BY

DEPT DATE

b.) Measured N values in ML

– ML not encountered

III.) MW-26/26A

a.) Aquitard Thickness (CL + ML)

$H = 9' + 32' = 41'$

ML (14' to 23') — see pg 31

CL (23' to 55') — see pg 33

b.) Measured N values in ML

$N = 18, 45$

Select thickness of CL + ML Aquitard to be used in Conceptual Design Subsurface Profile.

– In summary from pgs 23 + 35

$H_{CL+ML} = 32.5'$ (MW-24A)

$H_{CL+ML} = 23'$ (MW-25A)

WESTON SOLUTIONS

SHEET 36 of ___

CLIENT/SUBJECT ___ W.O. NO. ___

TASK DESCRIPTION ___ TASK NO. ___

PREPARED BY ___ DEPT ___ DATE ___

MATH CHECK BY ___ DEPT ___ DATE ___

METHOD REV. BY ___ DEPT ___ DATE ___

APPROVED BY ___ DEPT ___ DATE ___

$H_{CL+ML} = 41'$ (MW-26/26A)

- Averaging all 3 values:

$H_{Ave} = 32.2$

- Averaging only the first 2 values consistent with the logic that MW-24A + MW-25A are closest in distance from borings 29N/15E + 29N/20E which were used to develop the remainder of the subsurface profile:

$$H_{AVE} = \frac{32.5 + 23}{2} = 27.75'$$

say 28'

- Select $H_{AQUITARD} = 28'$ since a thinner aquitard thickness is cons. WRT the various completed stability analyses.

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CLIENT/SUBJECT W.O. NO.

TASK DESCRIPTION TASK NO.

PREPARED BY DEPT DATE

MATH CHECK BY DEPT DATE

METHOD REV. BY DEPT DATE

APPROVED BY

DEPT DATE

– Since $H_{AQUITARD} = 28'$ &

$H_{CL} = 8'$ (see pg 19):

$$H_{ML} = 28' - 8' = 20'$$

Select c, ϕ & γ_T for ML soils:

– "N" values in ML stratum

N = 19, 16 (Boring MW-24A)

N = 18, ~~45~~ (Boring MW-26/26

Note: no ML enc in MW-25A

Also, N = ~~30~~ was measured in ML soils in water Boring 29N/15E

– Disregarding N = 45 & 30 values as being anomalously high (cons.), use

$$N_{rep} = N_{ave} = \frac{19+16+18}{3} = 17.7 \rightarrow \text{say } 17.5$$

CLIENT/SUBJECT W.O. NO.

TASK DESCRIPTION TASK NO.

PREPARED BY DEPT DATE APPROVED BY

MATH CHECK BY DEPT DATE

METHOD REV. BY DEPT DATE DEPT DATE

- Assuming undrained cohesive soil behavior in the ML soils:

$c = c_u > 0$

$\phi = \phi_u = 0°$

& from pg 21 Figure:

$q_u = 1.25$ TSF

$\therefore C_u = \frac{q_u}{2} = \frac{1.25 \text{ TSF}}{2} = .625$ TSF

$= 1250$ psf

- From pg 17 Table, for "fine grained soils" with $N_{ref} = 17.5$ (say 17), $\gamma_T = 130.5$ pcf

6.) Aquifer

- Material consists of SP, SC & SM soils according to the USCS

CLIENT/SUBJECT W.O. NO.

TASK DESCRIPTION TASK NO.

PREPARED BY DEPT DATE

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APPROVED BY

DEPT DATE

- Assume material extends to great depth for the analyses
- From pgs 13→15, aquifer was not penetrated by water borings 29N/15E & 29N/20E. Therefore, use shoreline borings MW-24A, MW-25A & MW-26/26A to determine N_{rep} & c, ϕ, γ_T for this stratum
- From pgs 24→34:

$N = 18$ (SC) } MW-24A
$N = 32$ (SP) } MW-24A
$N = 24$ (SP) - MW-25A
$N = 35$ (SM) - MW-26/26A

- Cons. select $N_{rep} = 18$ bpf

CLIENT/SUBJECT ______ W.O. NO. ______

TASK DESCRIPTION ______ TASK NO. ______

PREPARED BY ______ DEPT ______ DATE ______

MATH CHECK BY ______ DEPT ______ DATE ______

METHOD REV. BY ______ DEPT ______ DATE ______

APPROVED BY ______ DEPT ______ DATE ______

– For granular soil behavior

$c = 0$ psf

$\phi > 0°$

& $\phi = \sqrt{20N} + 15°$

$= \sqrt{20 \times 18} + 15° = 34°$

& since Nrep was based on shoreline borings data in which the aquifer soils can reasonably be assumed to have been consolidated & by a greater overburden weight (i.e. ground surface @ elev 605'±) as compared to a lesser overburden weight @ the conceptual design subsurface profile location (i.e. approx 2500N coordinate line) where free water elev = 602' &

rength gained

CLIENT/SUBJECT ____ W.O. NO. ____

TASK DESCRIPTION ____ TASK NO. ____

PREPARED BY ____ DEPT ____ DATE ____ APPROVED BY

MATH CHECK BY ____ DEPT ____ DATE ____

METHOD REV. BY ____ DEPT ____ DATE ____ DEPT ____ DATE ____

by sediment surface elev = 594.5',
reduce calc. ϕ by 3° to
account for this effect

$\therefore$ reduce ϕ to $34° - 3° = 31°$

- From pg 17, for $N_{REP} = 13$ (see below)
& "coarse grained soils"
classification & using a revised N_{REP}
of: $\phi = \sqrt{20N} + 15°$

$$31° = \sqrt{20N} + 15°$$

$$N_{REP} = N = \frac{(31° - 15°)^2}{20} = 12.8$$

$\Downarrow$ say 13

$\therefore \gamma_T = 113$ pcf

In summary, the Developed Conceptual Design Subsurface Profile is shown on pg 42

SHEET 42 of

CLIENT/SUBJECT W.O. NO.

TASK DESCRIPTION CONCEPTUAL DESIGN SUBSURFACE PROFILE TASK NO.

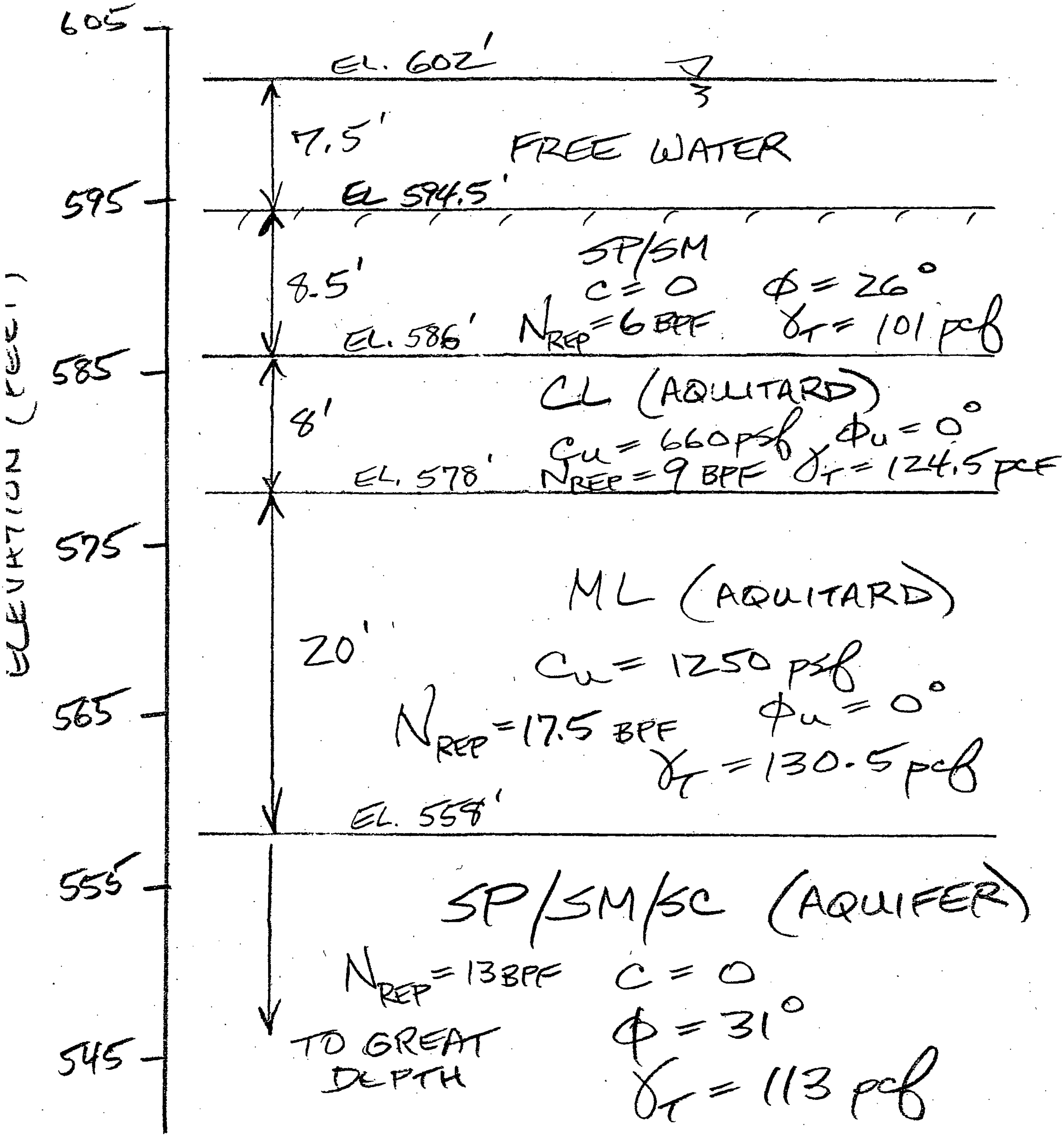
PREPARED BY DEPT DATE

MATH CHECK BY DEPT DATE

METHOD REV. BY DEPT DATE

APPROVED BY

DEPT DATE



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APPENDIX B

PRELIMINARY SHEET PILING DESIGN

WESTON SOLUTIONS

CLIENT/SUBJECT Ashland Wi Superfund Site W.O. NO.

TASK DESCRIPTION TASK NO.

PREPARED BY WLD DEPT DATE 11/12/09 APPROVED BY

MATH CHECK BY DEPT DATE

METHOD REV. BY DEPT DATE DEPT DATE

Preliminary Sheet Piling Design

1.) A preliminary sheet piling structural design was completed as part of this study using the Conceptual Design Subsurface Profile presented on page 4 of Appendix A. This preliminary design was structured to determine the required depth of embedment of the assumed cantilevered sheets below the proposed bay sediments excavation depth, the corresponding tip elevation & length of the sheeting, the required sheeting section based on Section Modulus requirements, and the max. lateral deflection of the sheet

2.) Assumptions used in completing this analysis are:

a) The top of the sheeting will be set at elev. 605.3'

WESTON SOLUTIONS

SHEET 2 of

CLIENT/SUBJECT W.O. NO.

TASK DESCRIPTION TASK NO.

PREPARED BY DEPT DATE | APPROVED BY

MATH CHECK BY DEPT DATE

METHOD REV. BY DEPT DATE | DEPT DATE

above the bay water surface elevation of 602'.

b.) Following installation of the sheeting the free water within the sheeted area will be removed by pumping.

c.) This will be followed by excavation of the bay bottom sediments within the sheeted area to a 5' depth.

d.) A dewatering system (e.g. a system of lateral + header dewatering trenches cut into the excavation bottom or equivalent) will be installed which will be designed to lower the water table within the sheeted area to a max.

WESTON SOLUTIONS

SHEET 3 of ____

CLIENT/SUBJECT ____ W.O. NO. ____

TASK DESCRIPTION ____ TASK NO. ____

PREPARED BY ____ DEPT ____ DATE ____

MATH CHECK BY ____ DEPT ____ DATE ____

METHOD REV. BY ____ DEPT ____ DATE ____

APPROVED BY ____ DEPT ____ DATE ____

elevation which is 2' deeper than the finished excavation elevation.

Based on the above discussion the sheet piling preliminary design subsurface profile shown on page 4 was developed. This is followed by input and output files from the sheet piling design software package PROSHEET which was used for the analysis. Some of these PROSHEET pages were "marked up" (see pgs 5, 6, 9, 10, 11 & 16) to clarify the results of the PROSHEET printout.

WESTON SOLUTIONS

CLIENT/SUBJECT Ashland WI Superfund Site   SHEET 4 of ____
W.O. NO. ____

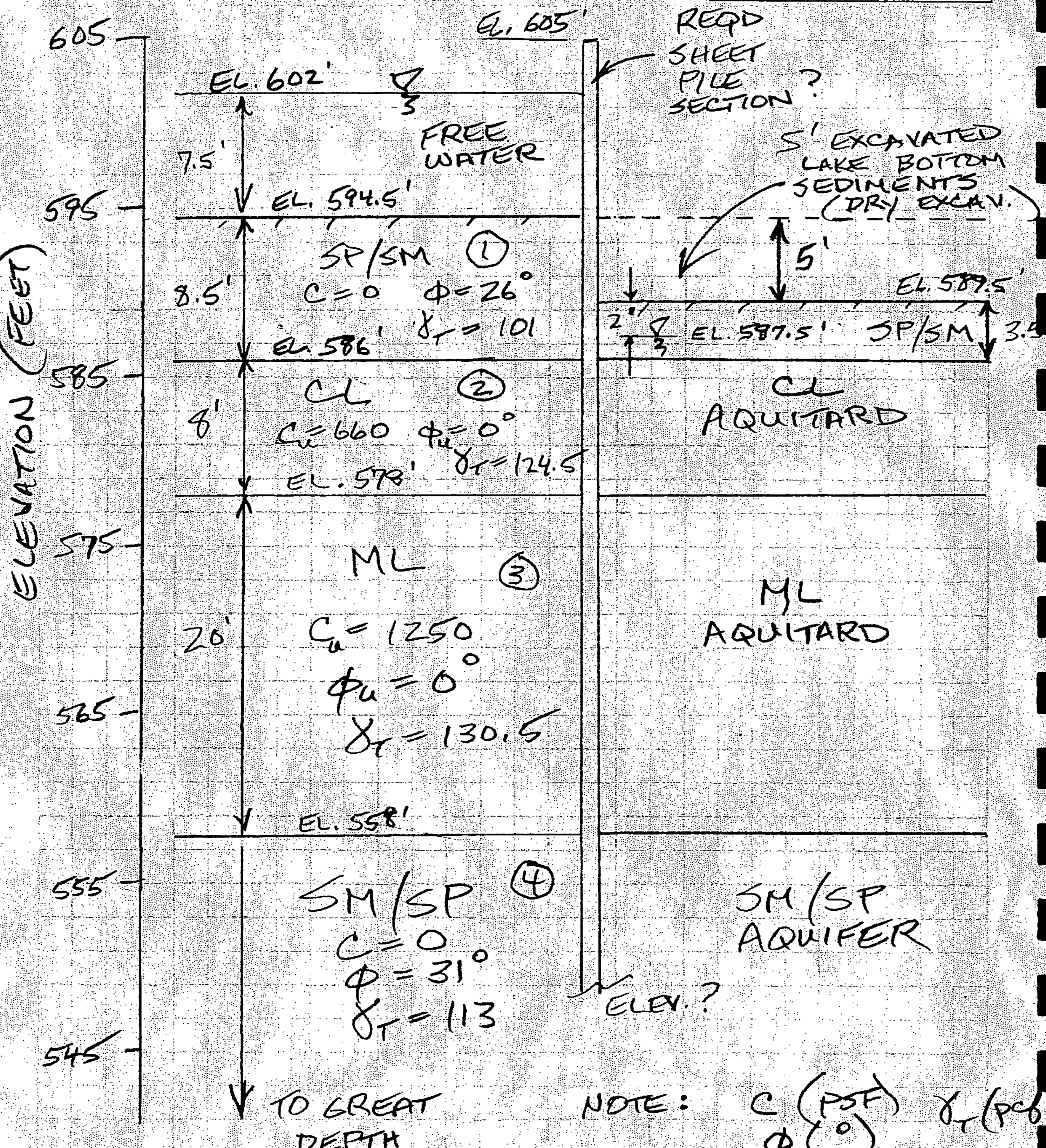
TASK DESCRIPTION Prelim. Sheet Piling Design   TASK NO. ____

PREPARED BY WLD   DEPT ____   DATE 11/12/09

MATH CHECK BY ____   DEPT ____   DATE ____

METHOD REV. BY ____   DEPT ____   DATE ____

APPROVED BY ____
DEPT ____ DATE ____



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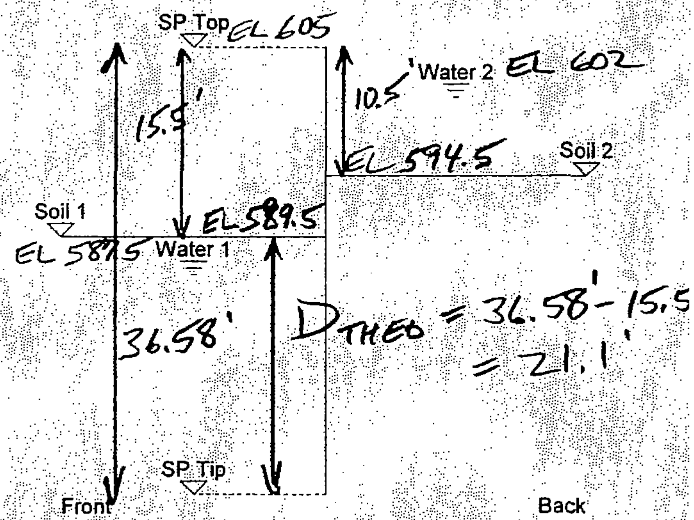
Geodata

	Unit
Sheet Pile Top Level [ft]	0.000
Sheet Pile Tip Level [ft]	36.583
Soil Level in Front [ft]	15.500
Soil Level behind [ft]	10.500
Anchorlevel [ft]	0.000
Water Level in Front [ft]	17.500
Water Level behind [ft]	3.000
Soil Surface Inclination in Front [Deg]	0.000
Soil Surface Inclination behind [Deg]	0.000
Caquot Surcharge in Front [kip/ft2]	0.000
Caquot Surcharge behind [kip/ft2]	0.000
Anchor Inclination [Deg]	0.000
Earth Support	Cantilever



1.) Required Sheet Pile Embedment Depth

a.) Theoretical:

$$D_{THEO} = 21.1'$$

b.) Conceptual Design:

$$D_{DESIGN} = (21.1')(\underbrace{1.3}_{FS}) = 27.4'$$

2.) Tip Elevation of Sheets (TE):

$$TE = EL.\ 605' - [15.5' + 27.4'] = 562.1$$

i.e. Approx. 4' above ML Aquitard / Aquifer interface

3.) Required Length (L) of Sheets:

$$L = 15.5' + 27.4' = 42.9'$$

$\downarrow$

use 45'

6

Soil Layers

Layers in Front

	Layer Tip [ft]	Density Moist [kip/ft3]	Density Submerged [kip/ft3]	Kph	Phi [Deg]	Delta [Deg]	Cohesion [kip/ft2]
Layer 1	19.000	0.101	0.039	3.710	26.000	-13.000	0.000
Layer 2	27.000	0.124	0.063	1.000	0.000	0.000	0.660
Layer 3	47.000	0.131	0.068	1.000	0.000	0.000	1.250
Layer 4	90.000	0.113	0.051	5.168	31.000	-15.500	0.000

Layers behind

	Layer Tip [ft]	Density Moist [kip/ft3]	Density Submerged [kip/ft3]	~~Kph~~ KAH	Phi [Deg]	Delta [Deg]	Cohesion [kip/ft2]
Layer 1	19.000	0.101	0.039	0.344	26.000	13.000	0.000
Layer 2	27.000	0.124	0.063	1.000	0.000	0.000	0.660
Layer 3	47.000	0.131	0.068	1.000	0.000	0.000	1.250
Layer 4	90.000	0.113	0.051	0.279	31.000	15.500	0.000

Assume $\delta = .5\phi$
+ for active side of wall
− for passive side of wall

7

Pile Section

Name	AZ14-770
Inertia [in4/ft]	170.620
Modulus [in3/ft]	25.203
Area [in2/ft]	6.213
Mass [lbs/ft2]	21.137
Steel Grade [lb/in2]	50000.000
Requested Safety	1.000

8

Extremal Values

	z Min [ft]	Min	z Max [ft]	Max
Deflection [ft]	0.000	-0.667	34.087	0.000
Cross Force [kip/ft]	34.087	-19.058	19.000	6.522
Moment [kipft/ft]	34.119	-0.832	26.349	68.235

9

Pile Check

		Depth [ft]
Name	AZ14-770	
Inertia [in4/ft]	170.620	
Modulus [in3/ft]	25.203	
Area [in2/ft]	6.213	
Mass [lbs/ft2]	21.137	
Steel Grade [lb/in2]	50000.000	
Minimal Moment [kipft/ft]	-0.832	34.119
M_{max} = Maxmimal Moment [kipft/ft]	68.235	26.349
Normal Forces at Max. Moment [kip/ft]	-0.358	34.119
Normal Forces at Min. Moment [kip/ft]	-0.358	26.349
Deflection at Min. Moment [ft]	0.000	34.119
Deflection at Max. Moment [ft]	-0.037	26.349
Min. Stress at Min. Moment [lb/in2]	-453.885	34.119
Max. Stress at Min. Moment [lb/in2]	338.582	34.119
Min. Stress at Max. Moment [lb/in2]	-32538.943	26.349
Max. Stress at Max. Moment [lb/in2]	32423.637	26.349
Safety > Req. Safety = 1.000	1.537	
Sheet Pile Top Level [ft]	0.000	
Sheet Pile Tip Level [ft]	36.583	
Sheet Pile Length [ft]	36.583	
Included OverLength [ft]	2.496	
Vertical Equilibrium [kip/ft]	-0.358	
Anchor Force (horiz.) [kip/ft]	0.000	

→ Select sheeting section based on M_{max}:

$M_{max} = S f_b$

S → section modulus of sheet

f_b → allowable flexural stress in steel

$f_b = .65 f_y$ → structural code requirement

Assume use of ASTM-572/GR 50 steel where $f_y = 50$ ksi (see pg 10)

$\therefore f_b = .65(50\text{ ksi}) = 32.5\text{ ksi}$

$$\therefore S_{REQD} = \frac{M_{max}}{f_b} = \frac{(68.235\ \text{kip-ft/ft})(12\ \text{in/ft})}{32.5\ \text{ksi}}$$

$$= 25.19\ \text{in}^3/\text{ft wall}$$

$\therefore$ Any section with $S_{ACT} > S_{REQD}$ is acceptable; e.g. for ARBED Steel Co AZ-14/770 is the most economical technically acceptable section (see pg 11

AZ

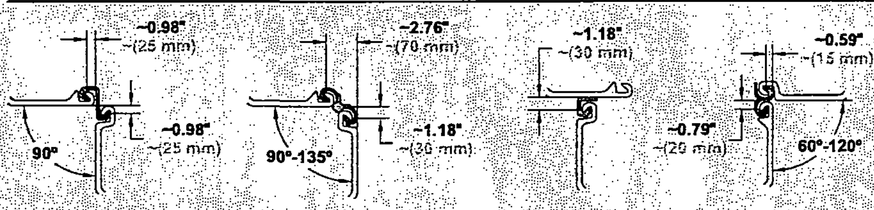
AZ Hot Rolled Steel Sheet Piling

(10)

Available Steel Grades								
AMERICAN			CANADIAN			EUROPEAN		
ASTM	YIELD STRENGTH		CSA G40.21	YIELD STRENGTH		EN 10248	YIELD STRENGTH	
	(ksi)	(MPa)		(ksi)	(MPa)		(ksi)	(MPa)
A 328	39	270	Grade 260 W	38	260	S 240 GP	35	240
A 572 Grade 42	42	290	Grade 300 W	43	297	S 270 GP	39	270
A 572 Grade 50	50	345	Grade 350 W	51	[illegible]	S 320 GP	46	315
A 572 Grade 55	55	380	Grade 400 W	58	400	S 355 GP	51	355
A 572 Grade 60	60	415				S 390 GP	57	390
A 572 Grade 65	65	450				S 430 GP	62	430
A 690	50	345				S 460 AP	67	460
A 690*	57	390						

*Not available for AZ 37–700 and larger.

Corner Piles



C 14
Weight: 9.68 lbs/ft
(14.4 kg/m)

Omega 18
Weight: 12.10 lbs/ft
(18.0 kg/m)

C 9
Weight: 6.25 lbs/ft
(9.3 kg/m)

Delta 13
Weight: 8.73 lbs/ft
(13.0 kg/m)

Delivery Conditions & Tolerances

	ASTM A 6		EN 10248	
Mass	± 2.5%		± 5%	
Length	+ 5 inches	– 0 inches	± 200 mm	
Height			[illegible]	
Thickness			≤ 8.5 mm	± 0.5 mm
			> 8.5 mm	± 6%
Width			± 2%	
Double Pile Width			± 3%	
Straightness			0.2% of the length	
Ends out of Square			[illegible] of the width	

Maximum Rolled Lengths*

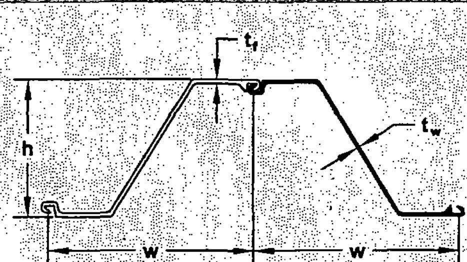
AZ	101.7 feet	(31.0 m)
C 9	59.1 feet	(18.0 m)
C 14	59.1 feet	(18.0 m)
Delta 13	59.1 feet	(18.0 m)
Omega 18	52.0 feet	(16.0 m)

* Longer lengths may be possible upon request.

AZ

AZ Hot Rolled Steel Sheet Piling

ArcelorMittal | skylinesteel

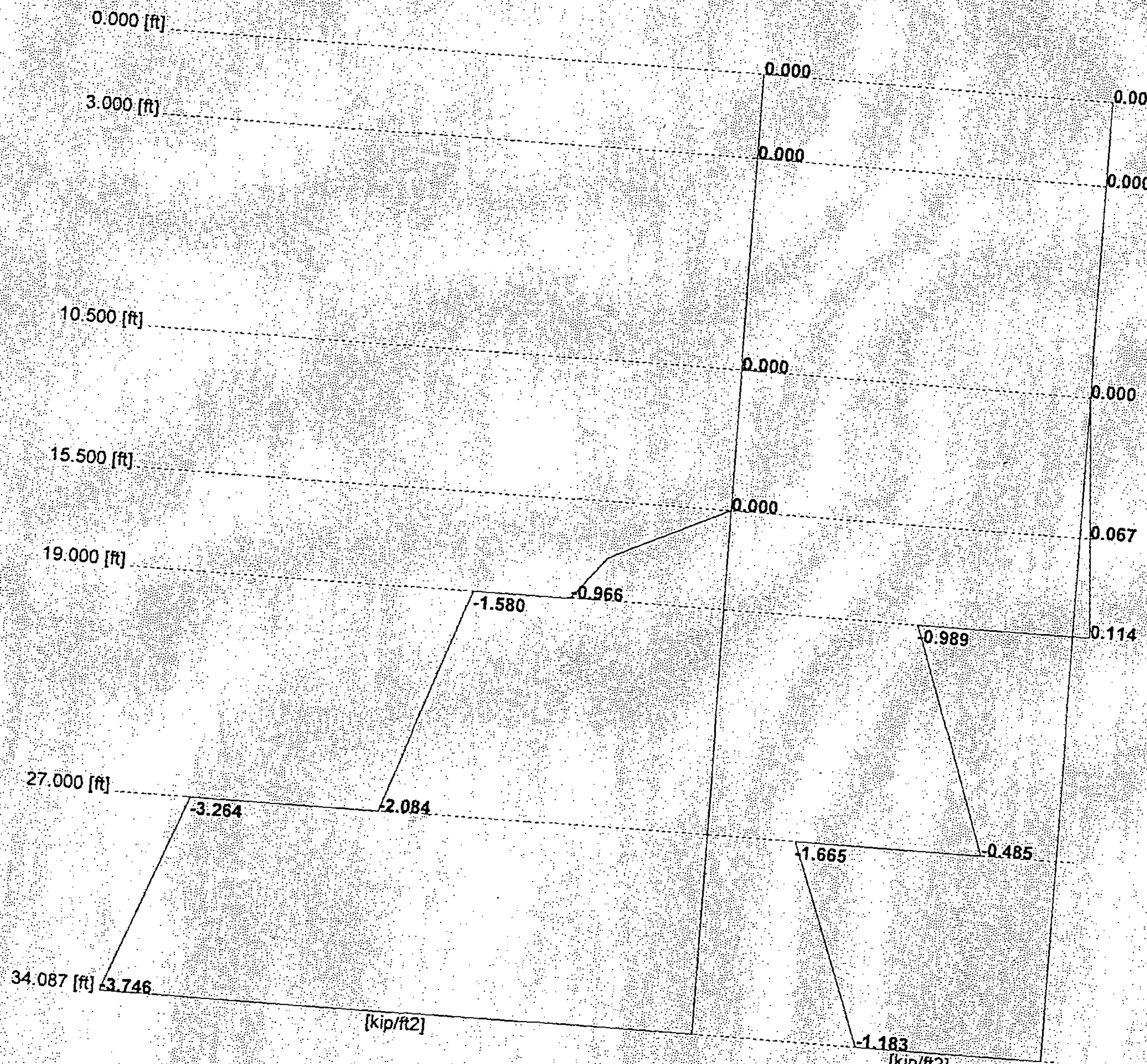


SECTION	Width (w) in	Height (h) in	THICKNESS Flange (t_f) in	Web (t_w) in	Cross Sectional Area in²/ft	WEIGHT Pile lb/ft	Wall lb/ft²	SECTION MODULUS Elastic in³/ft	Plastic in³/ft	Moment of Inertia in⁴/ft	COATING AREA Both Sides ft²/ft of single	Wall Surface ft²/ft²
AZ 12	26.38	11.89	0.335	0.335	5.94	44.42	20.22	22.3	26.2	132.8	5.45	1.23
AZ 13	26.38	11.93	0.375	0.375	6.47	48.38	22.02	24.2	28.4	144.3	5.45	1.23
AZ 14	26.38	11.97	0.413	0.413	7.03	52.62	23.94	26.0	30.7	156.0	5.45	1.23
AZ 12-770	30.31	13.52	0.335	0.335	5.67	48.78	19.31	23.2	27.5	156.9	6.10	1.20
AZ 13-770	30.31	13.54	0.354	0.354	5.94	51.14	20.24	24.2	28.8	163.7	6.10	1.20
AZ 14-770	30.31	13.56	0.375	0.375	6.21	53.42	21.14	25.2	30.0	170.6	6.10	1.20
AZ 17	24.80	14.92	0.335	0.335	6.53	45.96	22.24	31.0	36.2	231.3	5.64	1.35
AZ 18	24.80	14.96	0.375	0.375	7.11	49.99	24.19	33.5	39.1	250.4	5.64	1.35
AZ 19	24.80	15.00	0.413	0.413	7.74	54.43	26.34	36.1	42.3	270.8	5.64	1.35
AZ 17-700	27.56	16.52	0.335	0.335	6.28	49.12	21.38	32.2	37.7	265.3	6.10	1.33
AZ 18-700	27.56	16.54	0.354	0.354	6.58	51.41	22.39	33.5	39.4	276.8	6.10	1.33
AZ 19-700	27.56	16.56	0.375	0.375	6.88	53.76	23.41	34.8	41.0	288.4	6.10	1.33
AZ 25	24.80	16.77	0.472	0.441	8.74	61.49	29.74	45.7	53.4	382.6	5.91	1.41
AZ 26	24.80	16.81	0.512	0.480	9.35	65.72	31.79	48.4	56.9	406.5	5.91	1.41
AZ 28	24.80	16.85	0.551	0.520	9.97	70.15	33.94	51.2	60.5	431.6	5.91	1.41
AZ 24-700	27.56	18.07	0.441	0.441	8.23	64.30	28.00	45.2	53.5	408.8	6.33	1.38
AZ 26-700	27.56	18.11	0.480	0.480	8.84	69.12	30.10	48.4	57.1	437.3	6.33	1.38
AZ 28-700	27.56	18.15	0.520	0.520	9.46	73.93	32.19	51.3	60.9	465.9	6.33	1.38
AZ 37-700	27.56	19.65	0.669	0.480	10.68	83.46	36.33	68.9	79.2	676.6	6.76	1.46
AZ 39-700	27.56	19.69	0.709	0.520	11.34	88.63	38.59	72.5	83.7	714.0	6.76	1.46
AZ 41-700	27.56	19.72	0.748	0.559	12.00	93.74	40.84	76.2	88.3	751.4	6.76	1.46
AZ 46	22.83	18.94	0.709	0.551	13.76	89.10	46.82	85.5	98.5	808.8	6.23	1.63
AZ 48	22.83	18.98	0.748	0.591	14.48	93.81	49.28	89.3	103.3	847.1	6.23	1.63
AZ 50	22.83	19.02	0.787	0.630	15.22	98.58	51.80	93.3	108.2	886.5	6.23	1.63

Earth Pressure Diagram

Date: 11/11/2005

(12)



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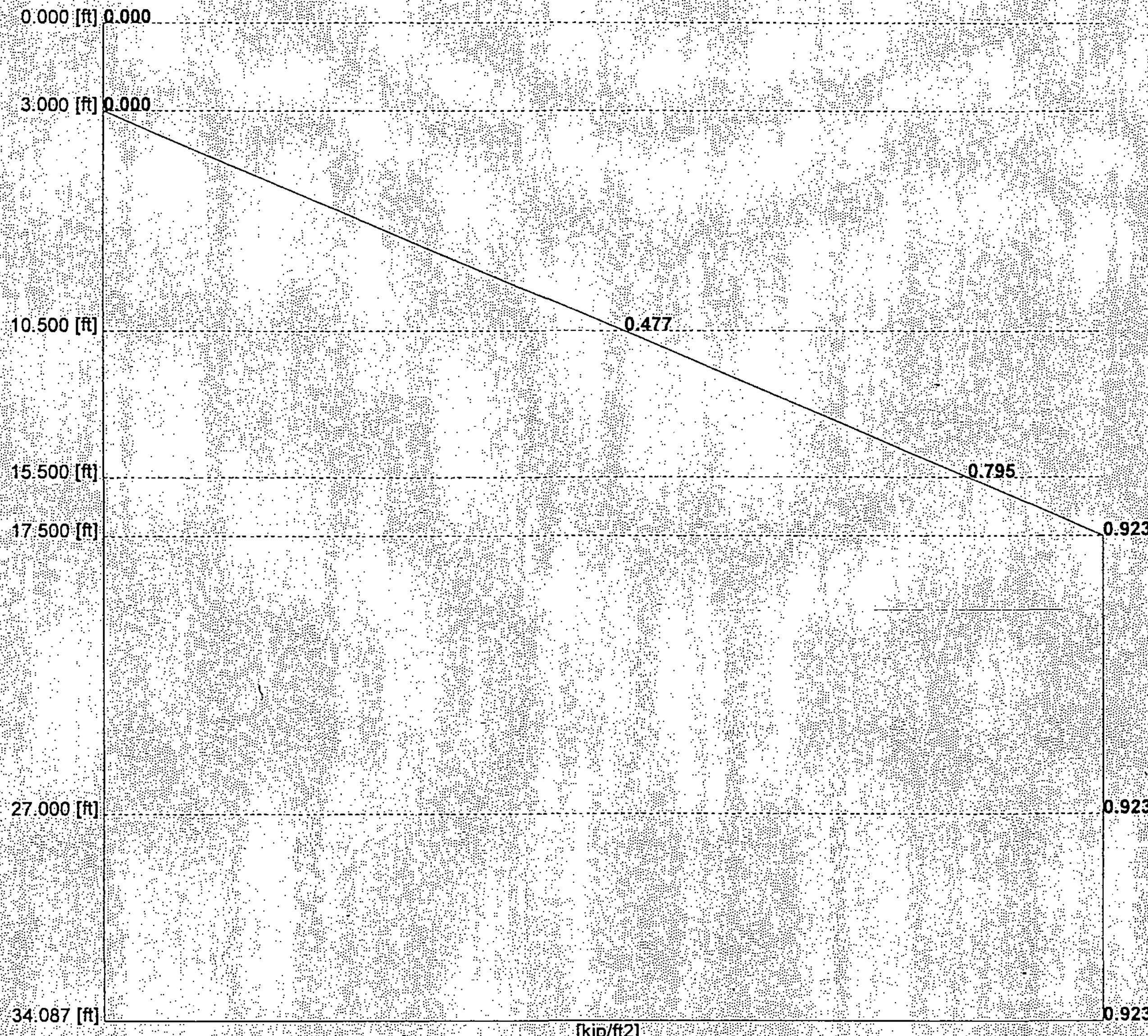
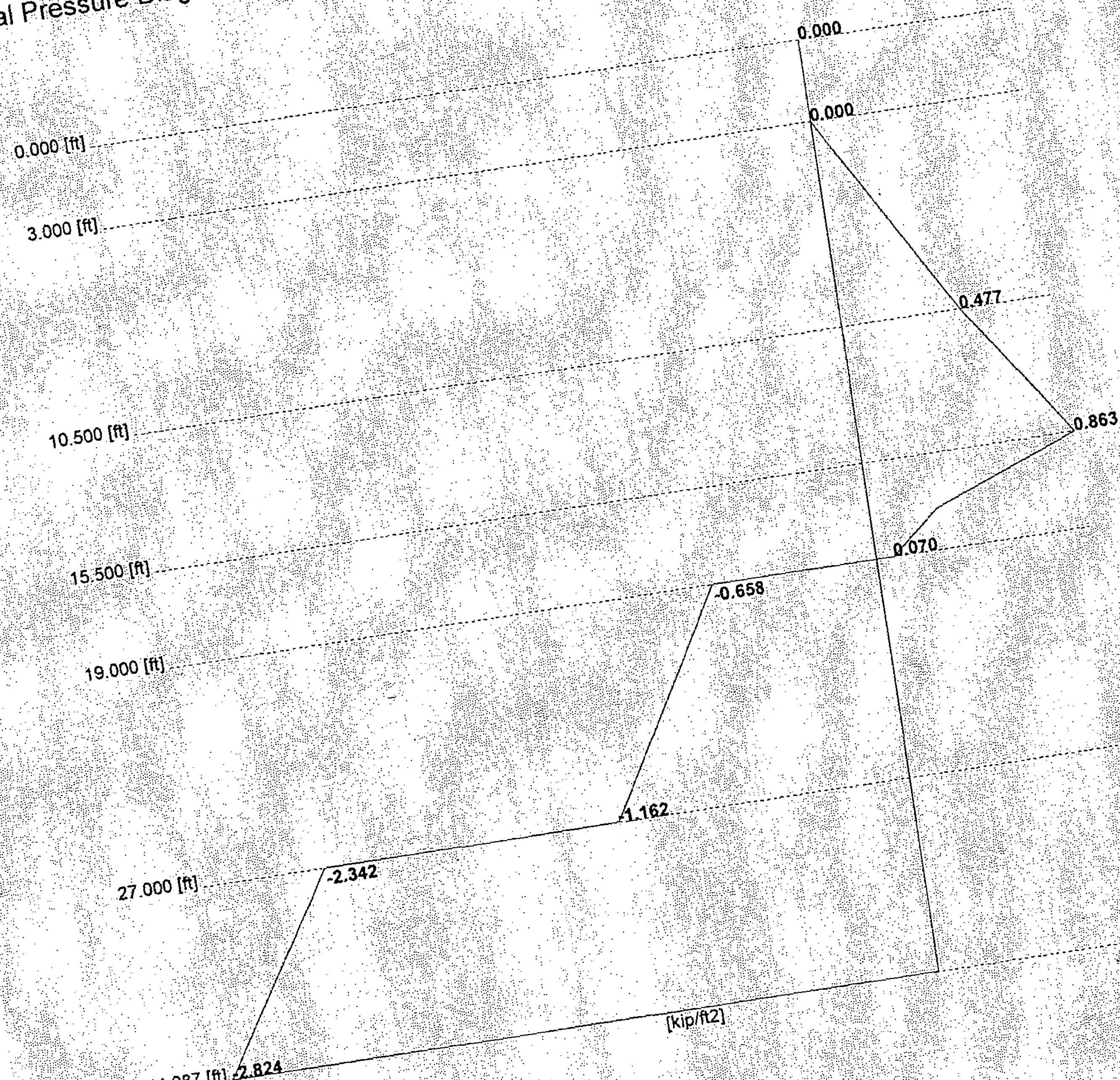
Water Pressure Diagram

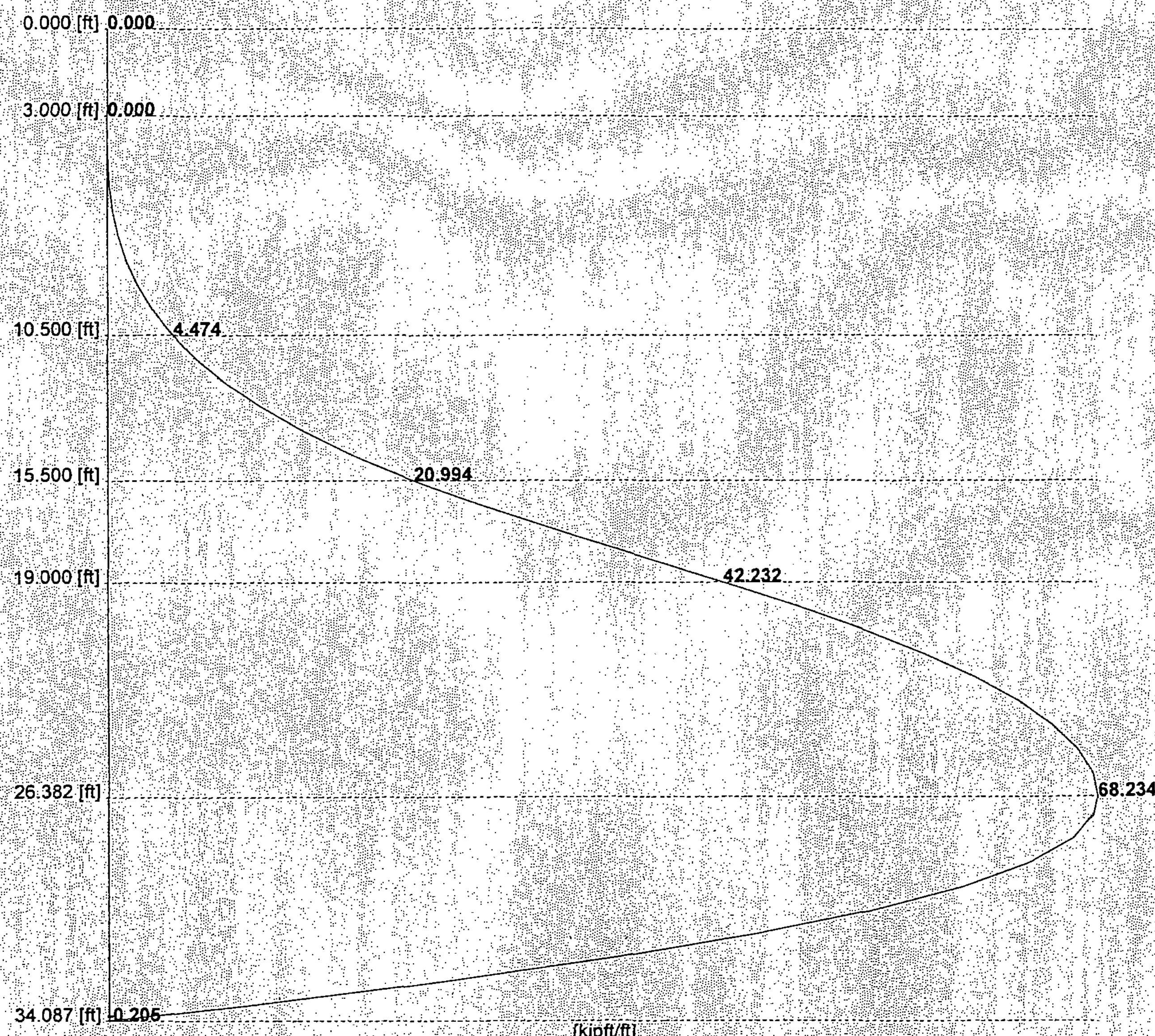


Total Pressure Diagram



Moment Diagram

15

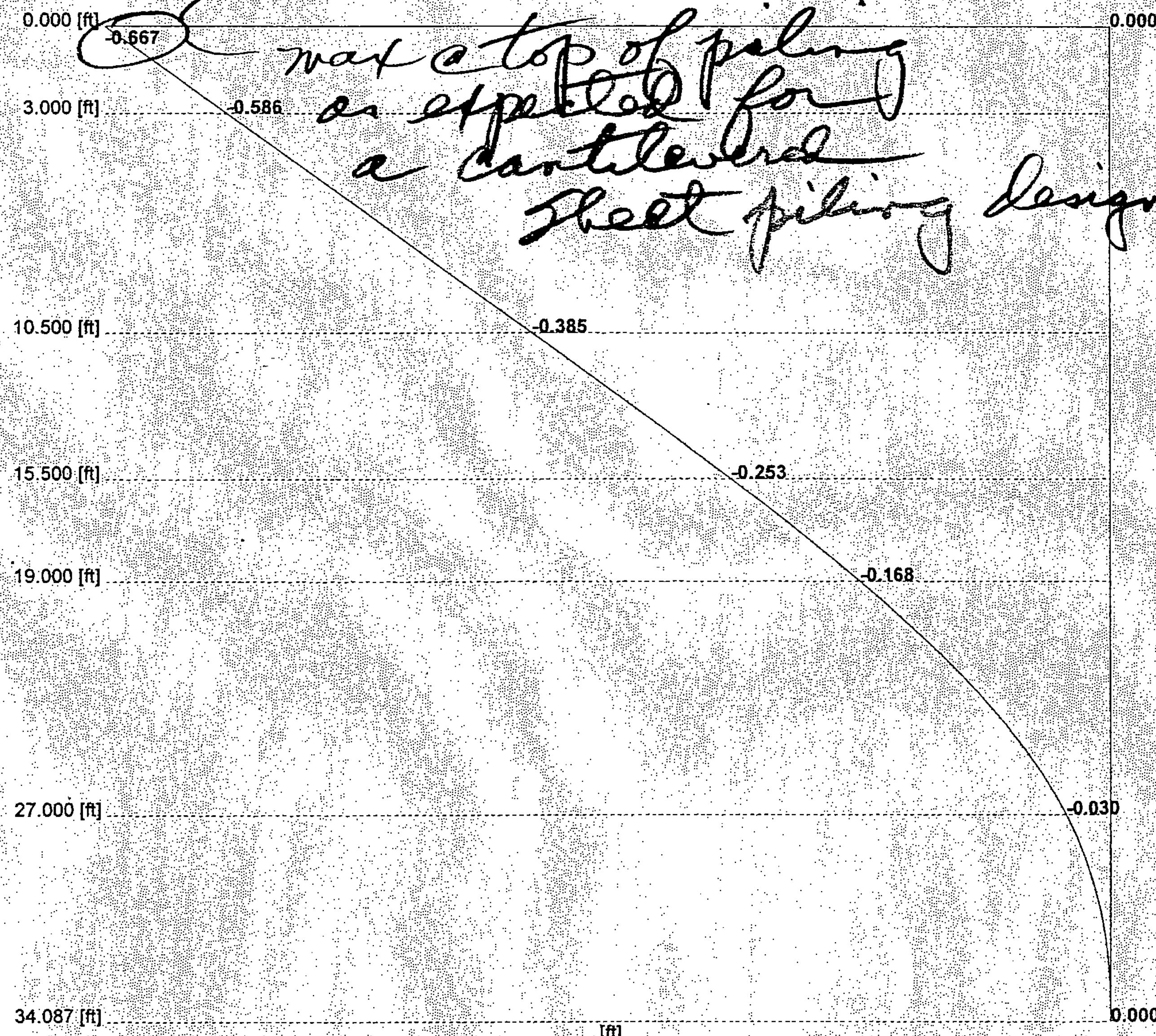


(16)

Deflection Diagram

= 8" < 12" OK

max @ top of piling as expected for a cantilevered sheet piling design



WESTON SOLUTIONS

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Cross Check of PROSHEET Selected Values of K_P + K_A

Ref: Fig 5a (see pg 16) from "Pile Buck Steel Sheet Piling Design Manual

I.) K_A (designated K_{AH} on Prosheet computer printout → see pg 3)

a.) Stratum 1

horizontal ground surface ← $\beta = 0° \rightarrow \therefore \beta/\phi = 0$

$\phi = 26°$

$\delta = .5\phi = 13°$ (assumed)

$K_A = .370$ ($\approx .344$)

see pg 3

PROSHEET Selected value

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REDUCTION FACTOR (R) OF K_p FOR VARIOUS RATIOS OF $-\delta/\phi$

ϕ \ δ/ϕ	-0.7	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	-0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.276	.221	.174

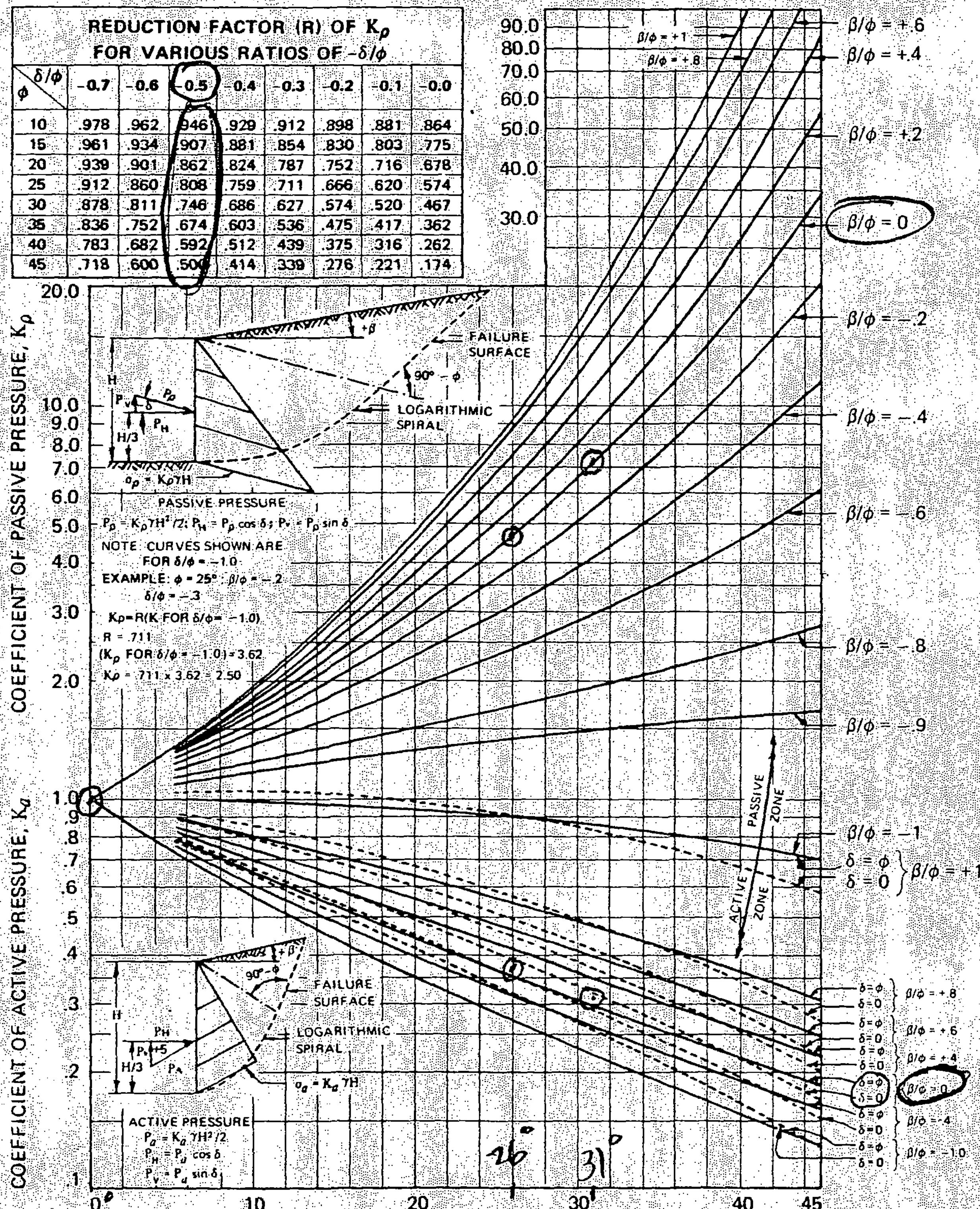

Fig. 5(a) — Active and passive coefficients with wall friction and sloping backfill (after Caquot and Kerisel[8])

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b.) Stratum 2

$\beta = 0° \rightsquigarrow \therefore \beta/\phi = 0$

$\phi = 0°$

$K_A = 1.0 \quad (=1.0)$ see pg 3

c.) Stratum 3

$\beta = 0° \rightsquigarrow \therefore \beta/\phi = 0$

$\phi = 0°$

$K_A = 1.0 \quad (=1.0)$ see pg 3

d.) Stratum 4

$\beta = 0° \rightsquigarrow \therefore \beta/\phi = 0$

$\phi = 31°$

$\delta = .5\phi = 15.5° \quad (\delta/\phi = .5)$
(assumed)

$K_A = .310 \quad (\approx .279)$ → see pg 3

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II.) K_p (designated K_{PH} on Proasheet computer printout)

a.) Stratum (1)

$\beta = 0°$ → $\beta/\phi = 0$

$\phi = 26°$

$\delta = .5\phi = 13°$

uncorrected K_p → $K_{PU} = 4.7$

& $R = .796$ for $\delta/\phi = .5$ & $\phi = 26°$ by interpolation of pg 16 Table

reduction factor

$K_p = 4.7(.796) = 3.741$

(≈ 3.710) see pg 3

b.) Strata (2) + (3)

$\beta = 0°$ → $\beta/\phi = 0$

& $\phi = 0°$

$K_p(2) = K_p(3) = 1.$ (= 1.

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c.) Stratum 4

$\beta = 0^\circ \rightsquigarrow \beta/\phi = 0$

$\phi = 31^\circ$

$\delta = .5\phi$ (assumed) $= 15.5^\circ$

$\therefore K_{Pu} = 7.2$

& $R = .732$ for $\delta/\phi = .5$

& $\phi = 31^\circ$

by interpolation of pg 16 Table

$\therefore K_P = .732 (7.2) = 5.270$

$(= 5.168)$

See pg 3

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APPENDIX C

EXCAVTION BOTTOM UPHEAVAL ANALYSIS

CLIENT/SUBJECT Ashland, Wisagerford Site W.O. NO.
TASK DESCRIPTION TASK NO.
PREPARED BY WLD DEPT DATE 11/13/09 APPROVED BY
MATH CHECK BY DEPT DATE
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EXCAVATION BOTTOM UPHEAVAL ANALYSIS

- An excavation bottom upheaval has been completed by Foth Infrastructure & Environment as discussed & summarized in their 6/1/09 letter report. A copy of this report is presented as an Attachment to this package.
- The analysis quantified the potential for upheaval of the excavation bottom precipitated by the uplift pressure in the SP/SM/SC aquifer caused by vertical confinement of this stratum by the overlying CL/ML aquitard, which is turn has created artesian pressure in the aquifer.

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CLIENT/SUBJECT W.O. NO.

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- Foth has assumed that the artesian pressure at the top of the aquifer (ie at the aquitard/aquifer interface) can be quantified by an artesian head of 617.1 ft based on wells screened near the top of the aquifer.

- Foth's excavation bottom upheaval analysis was simplistic in nature as it simply quantified the ratio between downward soil overburden stress due to the soil weight between the finished excavation bottom elevation & the aquiclude/aquifer interface elevation, to upward hydrostatic water pressure created by the artesian head at the aquitard/aquifer interface.

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The ratio between these downward + upward pressures was used by Foth to define a Factor of Safety (FS). In equation form:

$$FS = \frac{\sum_{i=1}^{n} H_i \gamma_{Ti}}{p_u \text{ (ARTESIAN)}}$$

where n = # of soil layers between the bottom of the excavation + the aquitard/aquifer interface

H_i = thickness of soil layer i

γ_T = total (i.e. wet) unit weight of soil layer i

p_u = (elev 617.1 − elev aquitard/aquifer interface) γ_w

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- Foth used this eqn to develop a plot on pg A-7 of the Attachment in which the thickness of the aquitard was varied as the x axis coordinate, with the resulting calculated FS as the y axis coordinate so as to demonstrate the sensitivity of the FS value to the aquitard thickness. As expected, the thicker the aquitard, the higher the FS value. Based on this Figure, & assuming the "dry excavate to top of aquitard @ elev. 590 ft" scenario with γ_T = 130 pcf aquitard soils:

$H_{AQUITARD} \geq 36'$ for FS = 1.2

$H_{AQUITARD} \geq 55'$ for FS = 1.4

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- Since the available, but limited geotechnical data from in water borings or shoreline borings near the proposed dry excavation footprint indicate that aquitard thicknesses may be much lower in this aquifer, a modified excavation bottom upheaval analysis was completed by the writer which includes the cohesive shear strength of the soils along the vertical sidewalls of the shear failure mass which is assumed to be uplifted by the artesian pressure as an additional resisting force in the FS eqn. The manner in which this was accomplished is documented as follows:

Assumptions:

1.) The Conceptual Design Subsurface Profile presented as page 42 of the Appendix A package was used for the analysis.

2.) The uplifted failure mass was assumed to be rectangular (or square as a special case) in plan with dimensions B (ft) × L (ft).

3.) The value of p_u (ARTESIAN) was calculated as:

p_u (ARTESIAN) = (el 617.1' − elev 558')(62.4 pcf)
= 3688 psf

→ Aquitard/Aquifer interface

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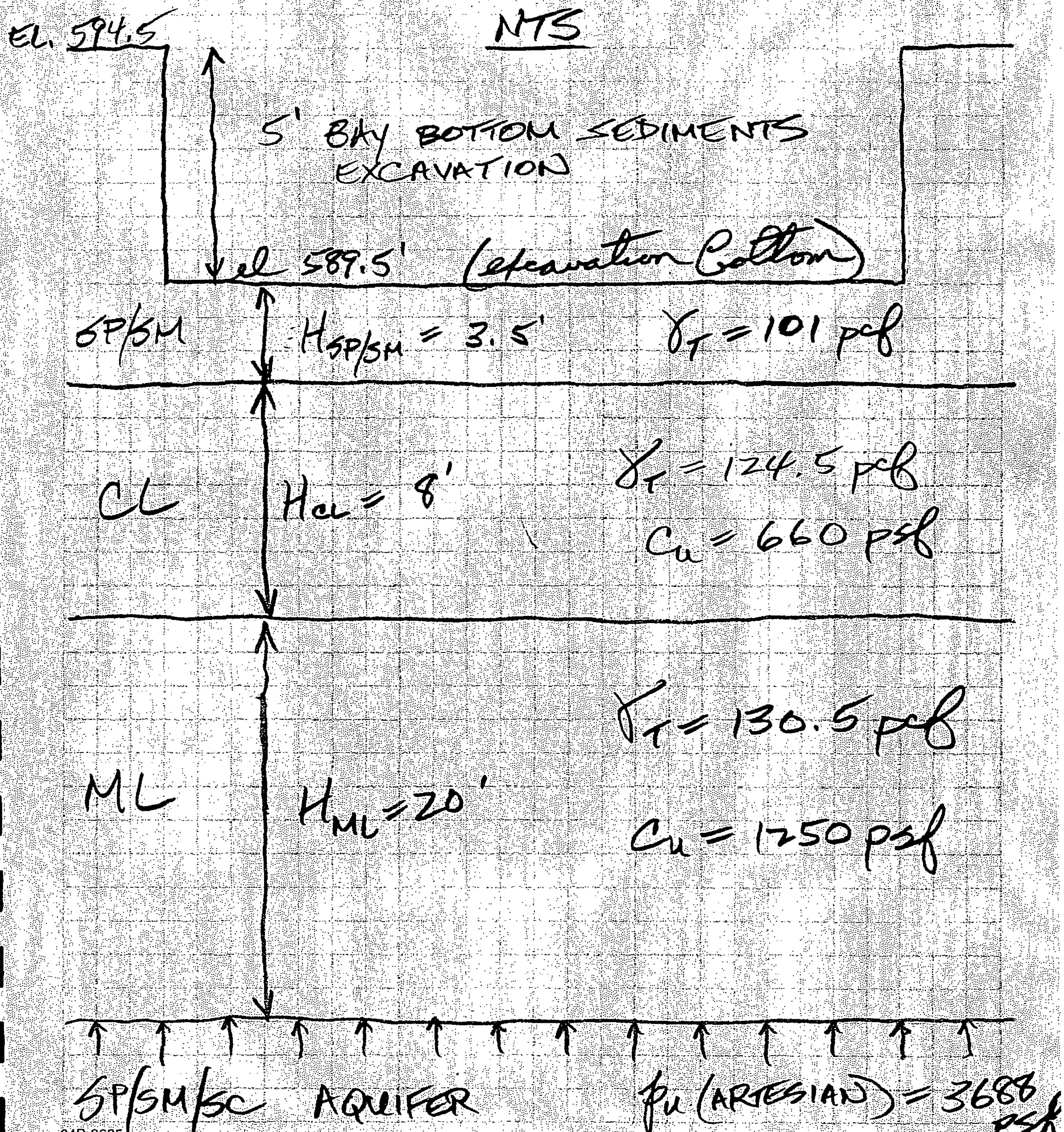

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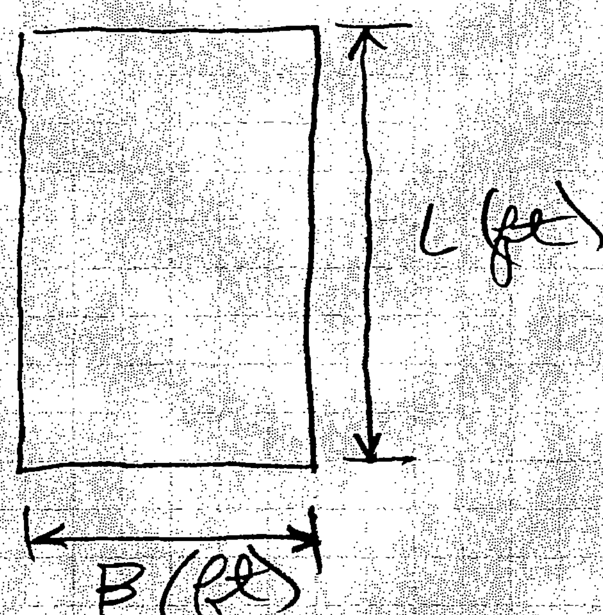
Plan:



To incorporate the vertical sidewall cohesive shear strength of the assumed rectangular solid failure mass into the analysis, the terms in both the numerator and denominator of the FS equation must be expressed in force units (i.e. lbs). The derivation of the governing equation is as follows:

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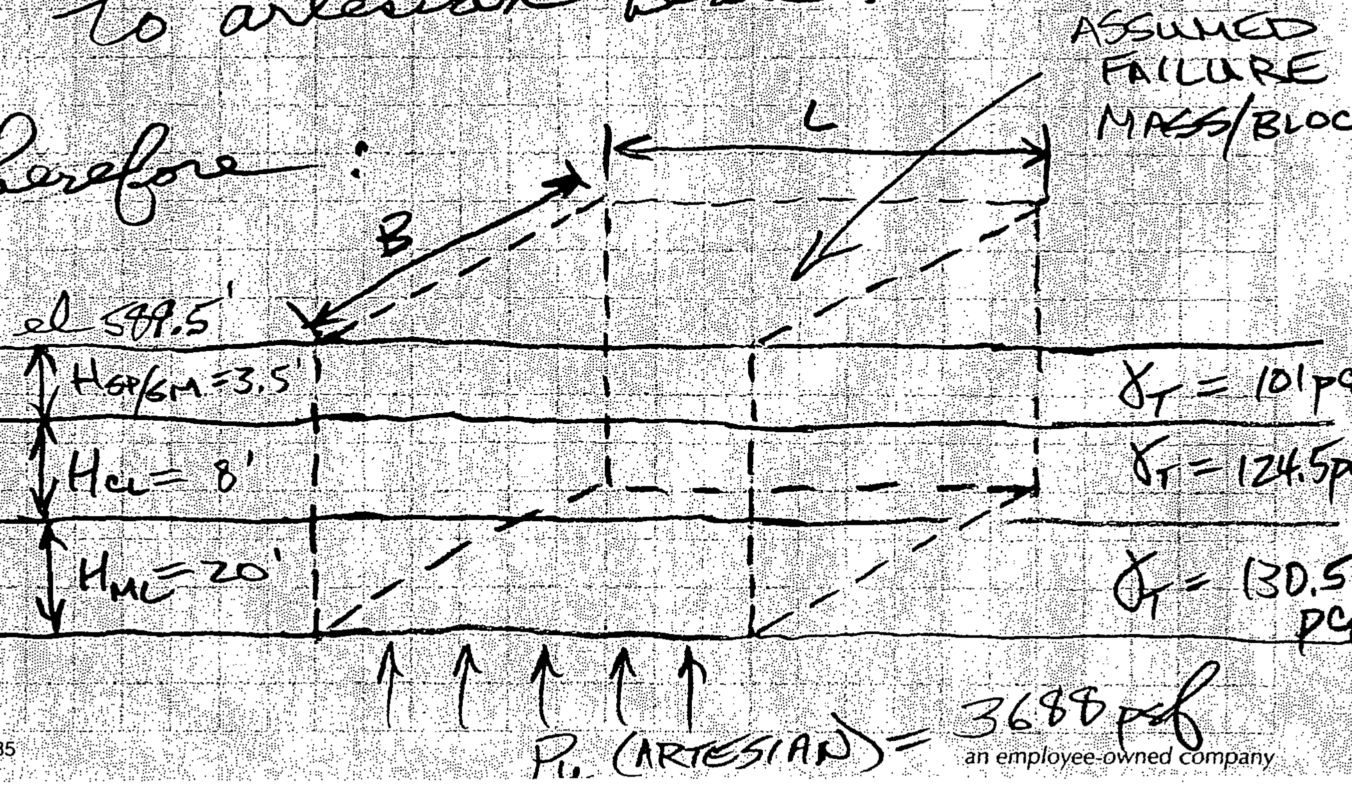
FS = Total weight of soil in B×L 3-D failure mass + Cohesive shear strength along assumed vertical shear failure planes of B×L plan dimensions in CL & ML strata

---

Total uplift force on base of B×L 3-D failure mass due to artesian head.

Therefore :



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$$\therefore FS = \Big[(3.5')(101\,pcf)(B)(L) + (124.5\,pcf)(8')(B)\ldots \xrightarrow{} lbs$$

Total wt of soil in B×L failure mass

$$+ (20')(130.5\,pcf)(B)(L)\Big] \xrightarrow{} lbs$$

cohesive shear strength (CL stratum) ⇐

$$+ \Big[(8')(B)(660\,psf)(2) + (8')(L)(660\,psf)(2)\ldots \xrightarrow{} lbs$$

cohesive shear strength (ML stratum) ⇐

$$+ \Big[20'(B)(1250\,psf)(2) + (20')(L)(1250\,psf)(\ldots$$

$$\overline{(3688\,psf)(B)(L)} \xrightarrow{} lbs$$

reducing

$$FS = \frac{3959.5\,BL + 19560(B+L) + 50{,}000(B+L)}{3688\,BL}$$

this equation was programmed in Microsoft EXCEL to investigate the sensitivity of the footprint size

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of the failure block. In particular, various combinations of B & L in which both variables were assumed to have values of 1', 10', 25', 50', 100', 150' + 200' were investigated. The results from the EXCEL spreadsheet are shown on pg 12. An inspection of this printout indicates that:

1) The larger the assumed plan size of the failure mass (BxL), the lower the FS value,

2) In order to provide a min FS = 1.25, the dry excavation cells should be no larger than B = 150', L = 200'

set by others as offset distance between shoreline + in water limit of sheet

Factor of safety for upheaval

Pressure Head (ft) 59.1

Soil Criteria:

Name	γ_T (pcf)	C_u (psf)	Thickness (ft)	Pressure (psf)
sp/sm	101	0	3.5	353.5
cl	124.5	660	8	996
ml	130.5	1250	20	2610
Total				3959.5

		L (ft)						
		1	10	25	50	100	150	200
B (ft)	1	33.92	19.14	18.15	17.82	17.66	17.60	17.58
	10	19.14	4.36	3.37	3.04	2.88	2.83	2.80
	25	18.15	3.37	2.39	2.06	1.89	1.84	1.81
	50	17.82	3.04	2.06	1.73	1.57	1.51	1.48
	100	17.66	2.88	1.89	1.57	1.40	1.35	1.32
	150	17.60	2.83	1.84	1.51	1.35	1.29	1.27
	200	17.58	2.80	1.81	1.48	1.32	1.27	1.24

FS > 1.25
when B = 150'
L = 200'
OK

FS < 1.25
when B = 200'
L = 200'
NG

Solve for a factor of safety for various B and L combinations.

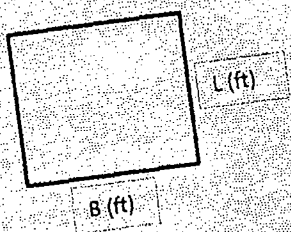

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Figure 2 of cover memo to these calculations shows this recommended dry excavation partitioned geometry in plan view.

Also, as noted on pg 14, FS against upheaval of excavation bottom should be ≥ 1.2.

(14)

As illustrated by the above two examples, the penetration depths computed with the simplified gross pressure method and the net pressure method are 5.6 and 5.76 m, respectively. The difference between the two is little. Therefore, the simplified gross pressure method is most commonly adopted.

5.7 Upheaval

If below the excavation surface there exists a permeable layer (such as sand or gravel soils) underlying an impermeable layer, the impermeable layer has a tendency to be lifted by the water pressure from the permeable layer. The safety, against upheaval, of the impermeable layer should be examined. As shown in Figure 5.31, the factor of safety against upheaval is

$$F_{up} = \frac{\sum_i \gamma_{ti} \cdot h_i}{H_w \cdot \gamma_w} \tag{5.17}$$

where

F_{up} = factor of safety against upheaval
γ_{ti} = unit weight of soil in each layer above the bottom of the impermeable layer
h_i = thickness of each soil layer above the bottom of the impermeable layer
H_w = head of the water pressure in the permeable layer
γ_w = unit weight of the groundwater.

The factor of safety against upheaval F_{up} should be larger than or equal to 1.2.

To safeguard the safety of excavation construction, the possibilities (of the occurrence) of upheaval at each stage of excavation should be analyzed. If drilling within the excavation zone is required (e.g. in order to place piezometers or build a well), the possible paths of water flow should be verified and the possible upheaval induced by drilling should be prevented to secure the excavation. Please see Section 5.8.2.

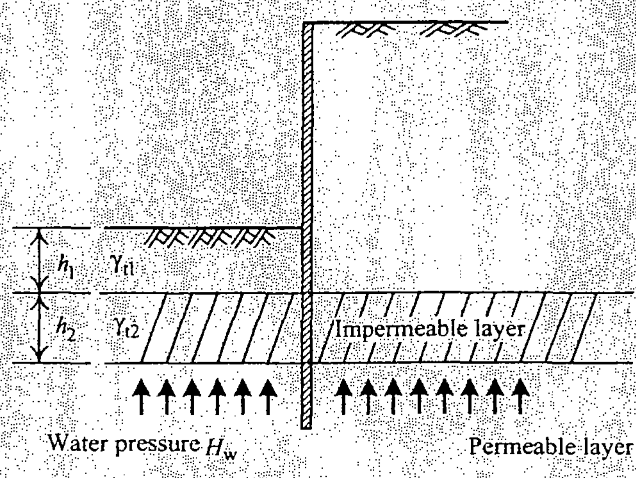

Figure 5.31 Analysis of upheaval.

Ref: "Deep Excavation / Theory & Practice"; Chang-Yu Ou

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ATTACHMENT

A-1

Foth

DRAFT Memorandum

June 1, 2009

TO: Jerry Winslow, Northern States Power Company

CC: Nick Azzolina, Steve Laszewski

FR: Jerry Eykholt, Jim Hutchison

RE: Preliminary Geotechnical Review – Sheet Pile Wall Installation for the Ashland/NSPW Lakefront Site

Background

This memorandum provides Foth's comments on the risks associated with basal heave failure and other geotechnical structural design elements for the two predominant sediment removal options for the Ashland/Northern States Power Company (NSPW) Lakefront Site. These two options currently include:

- Alternative SED-4B, removal by mechanical dredging, dewatering and off-site disposal (wet dredge), and
- Alternative SED-6B, hybrid remedy of a) excavation in the dry behind sheet pile using shore-based excavation techniques and equipment, and b) mechanical dredging for contaminated sediment further from the shore.

The sediment at the Site is underlain by the Miller Creek clay formation, which acts as an aquitard. There are known artesian conditions beneath the aquitard in the Copper Falls aquifer. Therefore, under certain removal conditions, uplift pressures from the artesian conditions at the base of the aquitard will exceed the overburden pressures. If the uplift forces that are not counter-balanced by overburden forces, then failure can result (basal heave failure), with risk to construction, project safety, and containment of contaminated sediments.

Foth has conducted an independent, preliminary geotechnical analysis and has generated a series of figures that reinforce and extend the calculations provided by AECOM.

Estimated Stresses for Initial Conditions and the Two Removal Options

For the simplest evaluation of basal heave, total downward vertical stresses on the base of the aquitard are compared to the uplift pressure. For the cases considered here, the uplift pressure is

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the pore pressure provided by the artesian head. The assumed geologic profile, total stresses and pore water pressures are shown in Figure 1.

The effective stress is the total vertical stress (from overburden) minus the pore pressure. Therefore, when the effective stress is negative, pore pressures are greater than overburden and there is the potential for uplift or a basal heave failure. The magnitude of negative effective stresses is largest for a dry/hybrid removal case at the top of the aquitard (elev. 590 ft.), at -750 psf (pounds per square foot), but are negative throughout the entire aquitard thickness. Immediately after removal, the clay cannot drain freely and pore pressures from the initial state would remain. This type of pore pressure-effective stress consideration is called the "undrained state", and it is often found to be a critical state in a more complete analysis of geotechnical stability.[1]

The situation of negative effective stresses over the whole aquitard causes concern with regard to basal failure. In contrast, the wet dredge removal scenario (SED-4B) yields a positive effective stress throughout the aquitard.

The severity of the unloading condition on the stability of the top of the aquitard depends on several factors, including the stiffness (shear strength), geometric factors related to the configuration of the excavation, and the hydraulic conductivity of the aquitard.

As mentioned above, the result of a negative effective stress at the top of the aquitard for the hybrid removal option is not unexpected. This result would occur even without artesian conditions, as is shown in Figure 2. Here, the head at the base of the aquitard is set at the lake elevation, and the pore pressure at the base of the aquitard is 2184 psf, nearly 1000 psf lower than the artesian condition. The effective stress at the top of aquitard is the same (-750 psf), but effective stresses increase to positive values at depth within the aquitard.

The factor of safety against basal heave failures was calculated for various values of aquitard thickness and unit weights (sediment and aquitard density) and plotted by AECOM. With the factor of safety defined as the ratio of the overburden stress to the porewater pressure at the base of the aquitard, Foth has reproduced the calculations and plotted curves for the same conditions. The result is shown in Figure 3. The overall agreement between the Foth calculations and the AECOM plot is good.

Discussion on Need for More Advanced Geotechnical Analysis for Wet and Dry Sediment Removal

Additional sediment and subgrade geotechnical characteristic data across the site could occur this summer, but should also include a basic framework of:

- Engage the agencies in the planning and evaluation of the key geotechnical issues,
- If new data are needed, include needs in a future work plan,

[1] Drained unloading, which would occur at the top of the aquitard after enough time is provided to relieve pore pressures, generally produces higher effective stresses.

A-3

- Conduct preliminary structural sheet pile design of removal alternatives prior to new data collection, and
- Present findings to agencies prior to ROD release

These new data can be collected using standard drilling techniques.

Any work plan for collecting these data should incorporate the need to confirm the issues related to basal heave risks (aquitard thickness, consistency and stability) as well as sediment and subgrade geotechnical characteristics associated with wet removal of sediment.

The AECOM plot shows a suitable range for factors of safety for basal heave to be 1.2 to 1.4, and that the aquitard thickness should be greater than ~35 feet for the safe conditions. The preliminary analysis and simple definition for basal stability is useful for identifying a potential problem. However, a more complete geotechnical analysis is needed to quantify factors such as shear strength of the aquitard and geometric factors related to the configuration of the excavation.

In addition to the basal failures from uplift, the presence of sand seams, cracks, or other conductive hydraulic features in the aquitard may cause seepage problems for various removal options. As with basal heave failures, the most difficulty is expected for the dry removal option, for which pressure gradients between the top and bottom of the aquitard are likely to be the highest.

Specific geotechnical data needs obtained from existing data, which may be used in the preliminary design should consist of the following:

- all available data on aquitard thickness and artesian head over site area,
- elevations of existing sediment, post-dredge sediment, top of aquitard,
- shear strength testing data on aquitard material,
- blow counts from available logs in area,
- review of any existing shear strength data on sands and aquitard clay materials, and
- careful review of boring logs over area for any presence of sand, cracks or other conductive features in the clay aquitard.

New data collection from borings could include:

- elevations of top of sediment and top and bottom of aquitard material,
- blow count and moisture content data with depth,
- shear strength data with depth of different strata encountered by the borings, and
- gradation, permeability and consolidated undrained shear strength data with depth.

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Foth

DRAFT Memorandum

Summary

A preliminary analysis of the potential for a basal heave failure has indicated that the dry removal option may result in a stress condition that is unstable. In particular, due to artesian conditions, the pore pressures may exceed the overburden pressures at the base of the aquitard. Since a basal failure during excavation carries significant safety and project risks, it may be prudent to remove from consideration this alternative from actively promoted remediation alternatives going forward.

In addition, other factors that affect basal stability, such as the shear strength of the aquitard clays and geometric factors associated with potential failure conditions may be considered in a more advanced analysis, if required. Further analyses should include review of historical and site geotechnical information (such as aquitard thickness, evidence of shear strength of the aquitard, and artesian heads) and possibly additional site borings.

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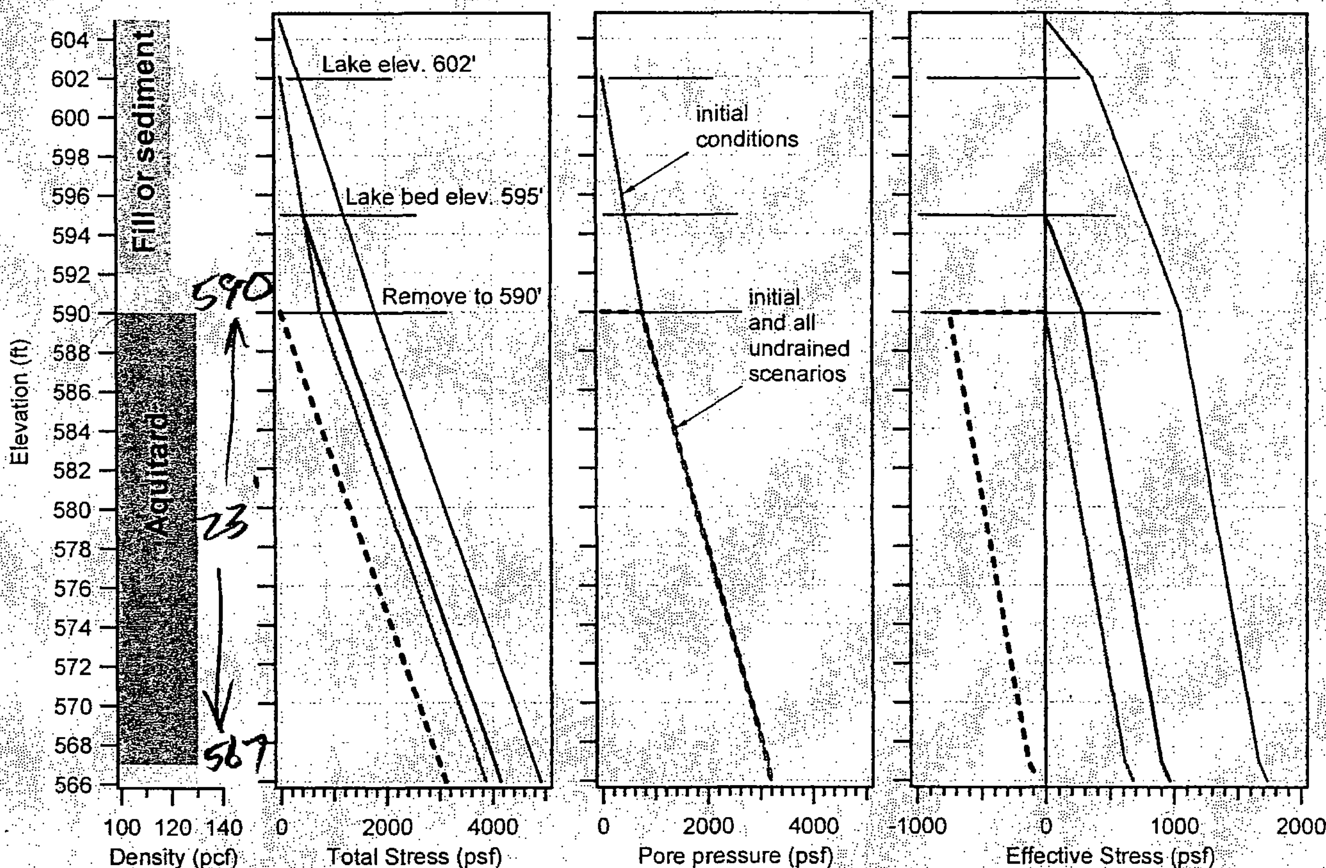
A-



Notes:

1. Assumes artesian conditions at base of aquitard.
   Head = 617.1 ft.
   pore water pressure = (62.4 pcf) (617.1 - 567 ft) = 3126 psf.
2. Assumes initial pore pressures in aquitard are in equilibrium.
3. Undrained cases assume that pore pressures in aquitard do not change.

Scenarios

Initial Conditions
—— A1: Land surface at Elev 605'
—— A2: In bay, sediment surface at 595'

Post-removal conditions (undrained)
---- B2: in bay, after dry excavation to 590'
—— B3: After dredging to 590' (wet removal)

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FIGURE 1

ASHLAND PROJECT
ESTIMATED EFFECTIVE STRESSES IN MILLER CREEK AQUITARD WITH VARIOUS REMOVAL SCENARIOS

Scale: AS SHOWN		Date: MAY 2009
Drawn By: GRE	Checked By: JBM	Scope: 09X001

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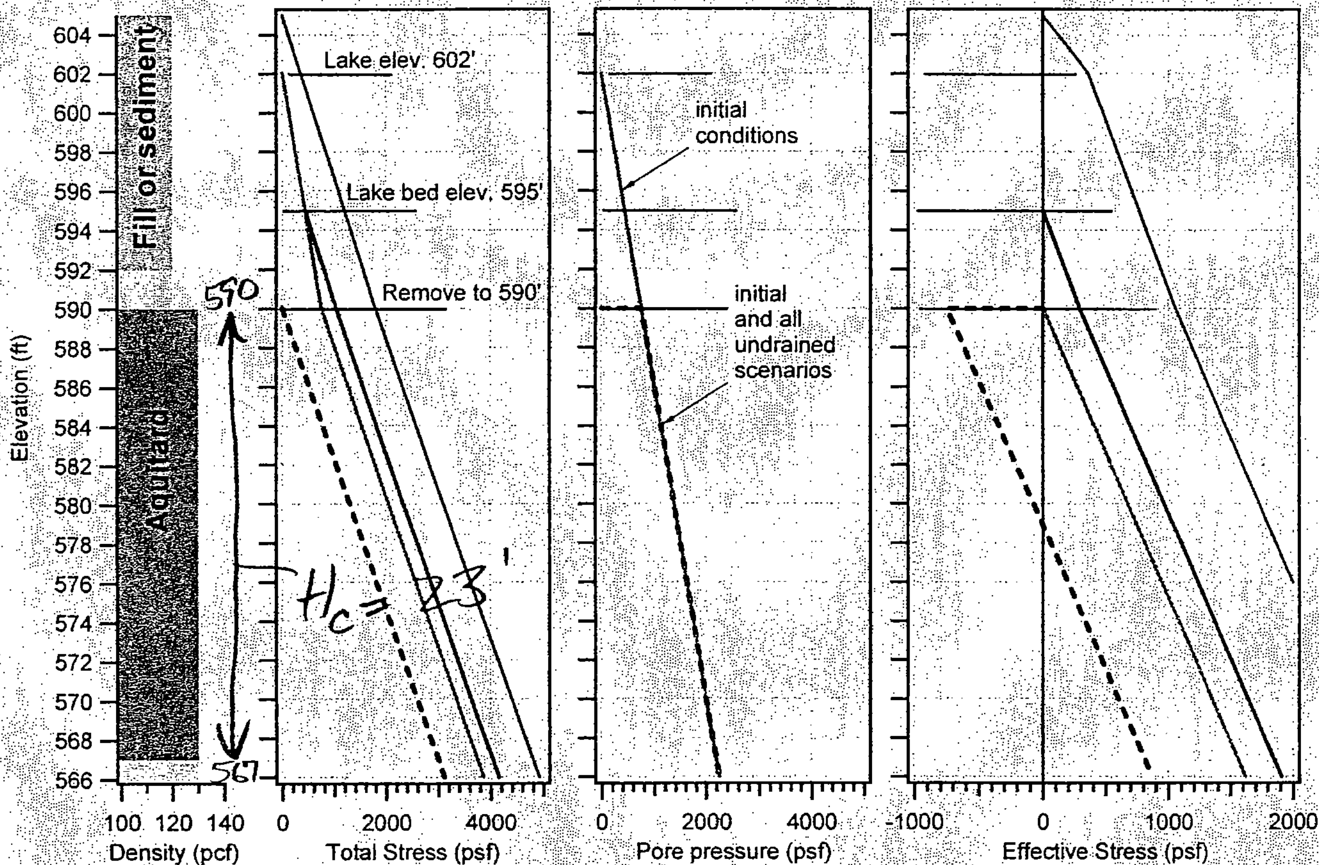

Notes:

1. Assumes artesian conditions removed at base of aquitard.
   Head = 602 ft.
   pore water pressure = (62.4 pcf) (602 - 567 ft) = 2184 psf.

2. Assumes initial pore pressures in aquitard are in equilibrium.

3. Undrained cases assume that pore pressures in aquitard do not change.

Scenarios:

Initial Conditions
—— A1: Land surface at Elev 605'
—— A2: In bay, sediment surface at 595'

Post-removal Conditions
---- B2: in bay, after dry excavation to 590'
—— B3: After dredging to 590' (wet removal)

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FIGURE 2

ASHLAND PROJECT
ESTIMATED EFFECTIVE STRESSES IN MILLER CREEK AQUITARD WITH REMOVAL OF ARTESIAN CONDITION

Scale: AS SHOWN		Date: MAY 2009
Drawn By: GRE	Checked By: JBM	Scope: 09X001

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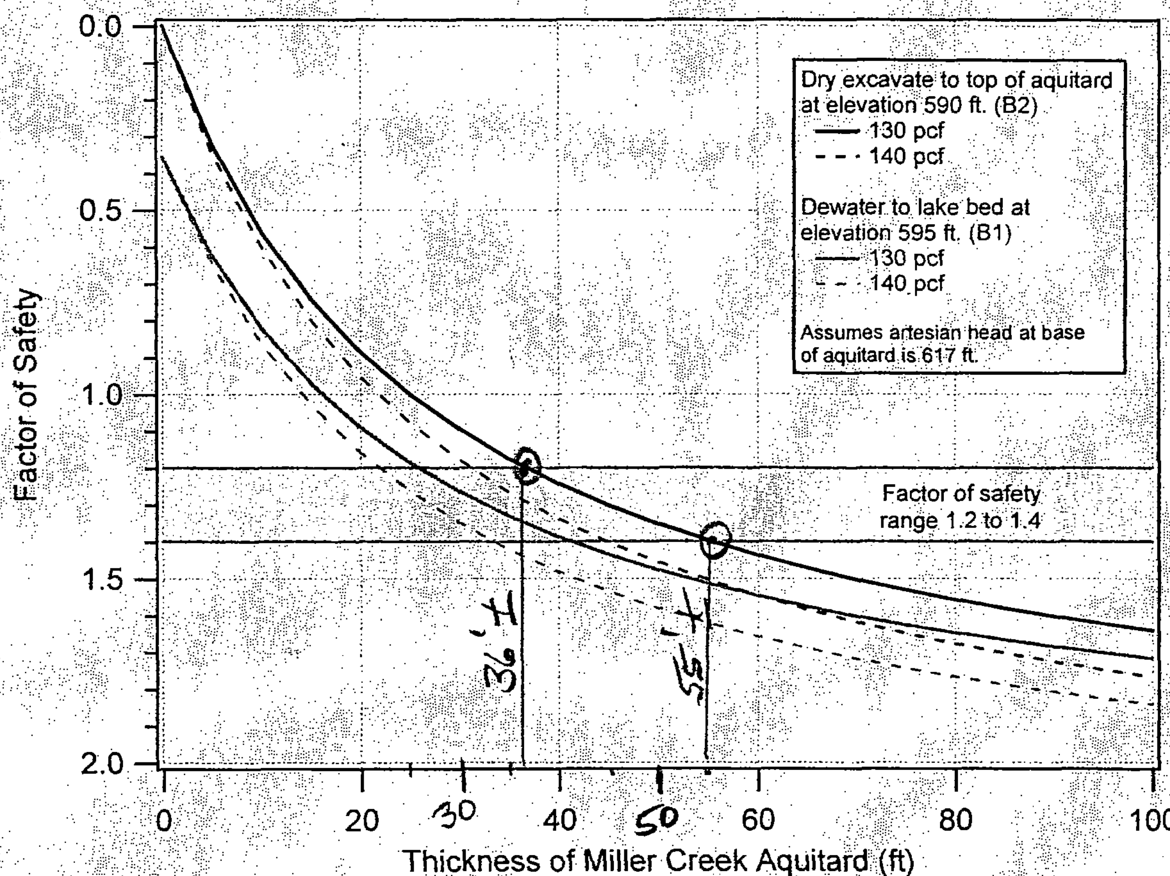

Notes:

1. Factor of safety is for simple basal heave, the ratio of total vertical overburden pressure to artesian pressure at base of the aquitard.

   Example calculation (dry excavation case):

   Aquitard thickness = 37 ft., unit weight = 130 pcf
   Elevation at base of aquitard = 553 ft.

   overburden pressure = (37 ft.) (130 pcf) = 4810 psf
   artesian pressure = (617 - 553 ft) (62.4 pcf) = 3994 psf

   FS = 4810 psf / 3994 psf = 1.20

2. Analysis does not consider resistance to basal heave due to shear strength of aquitard clay, and it ignores geometric effects of potential failure surfaces. Analysis should be considered as preliminary.
3. Other failure mechanisms not considered here, such as from piping.
4. Figure is an independent analysis and check of analysis provided by AECOM, received by Foth on 5/21/09. The agreement is excellent.

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FIGURE 3

ASHLAND PROJECT
PRELIMINARY ANAYSIS OF BASAL HEAVE
FAILURE FACTORS OF SAFETY FOR AQUITARD

Scale: AS SHOWN	Date: MAY 2009	
Drawn By: GRE	Checked By: JBM	Scope: 09X001

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APPENDIX D

BASAL HEAVE SHEAR FAILURE INSTABILITY ANALYSIS

WESTON SOLUTIONS

CLIENT/SUBJECT Ashland, Wi. Superfund Site SHEET 1 of 11

W.O. NO.

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PREPARED BY WLD DEPT DATE 11/12/09 APPROVED BY

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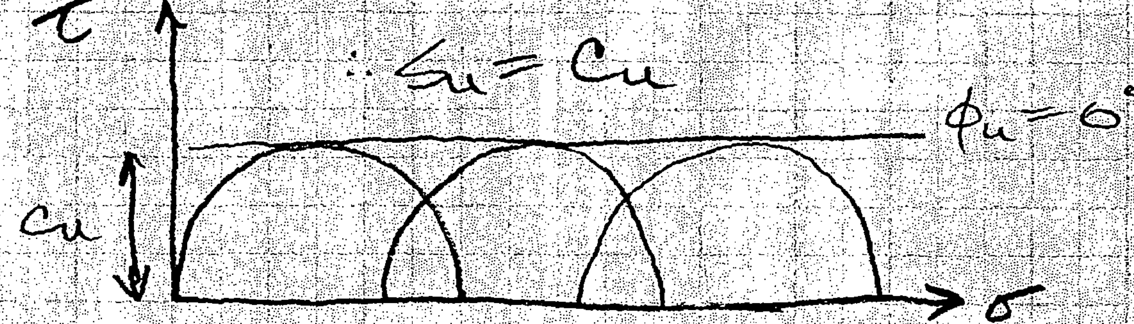
Basal Heave Shear Failure Instability Analysis

- Analysis based on discussion on pages 134 → 146 in "Deep Excavation" text by C.Y. Ou → see Attachment
- As discussed in this reference:
  i.) Basal heave is only relevant to fine-grained cohesive soils under undrained soil shear strength conditions. That is, clayey soils (e.g. CL & ML strata at the project site) with $c_u > 0$ & $\phi_u = 0$

That is:



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2.) Basal heave represents a true failure mechanism of the excavation bottom based on shear failure of the soil beneath & immediately adjacent to the excavation bottom. See Figure 5.10 of Attachment.

3.) Three (3) quantitative procedures are discussed to evaluate this potential. These are:

a.) Bearing Capacity (BC)
b.) Negative Bearing Capacity (NBC)
c.) Slip Circle

All calculate a Factor of Safety (F_s) against excavation bottom instability via shear failure in the underlying soils as the ratio between resisting forces and driving forces.

4.) The NBC analysis was selected for this evaluation because:

a.) As shown in Fig 5.12 of the Attachment, it generally predicts the lowest value of F_b as compared to the remaining 2 procedures when Su is constant with depth as assumed for the aquitard soils in this study.

b.) As discussed on pgs. 140 + 141 of the Attachment, the NBC procedure can be used

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for a 2 layer cohesive soil subsurface environment. This modification is referred to as the Bjerrum & Eide procedure as quantified by eqn 5.13 of the Attachment

c.) As discussed in the first full paragraph of page 146 of the Attachment, the NBC method in particular the Bjerrum & Eide's modification to NBC theory for 2 cohesive soil layers appear to yield reasonable results & are widely used.

The Bjerrum & Eide NBC analyses follows:

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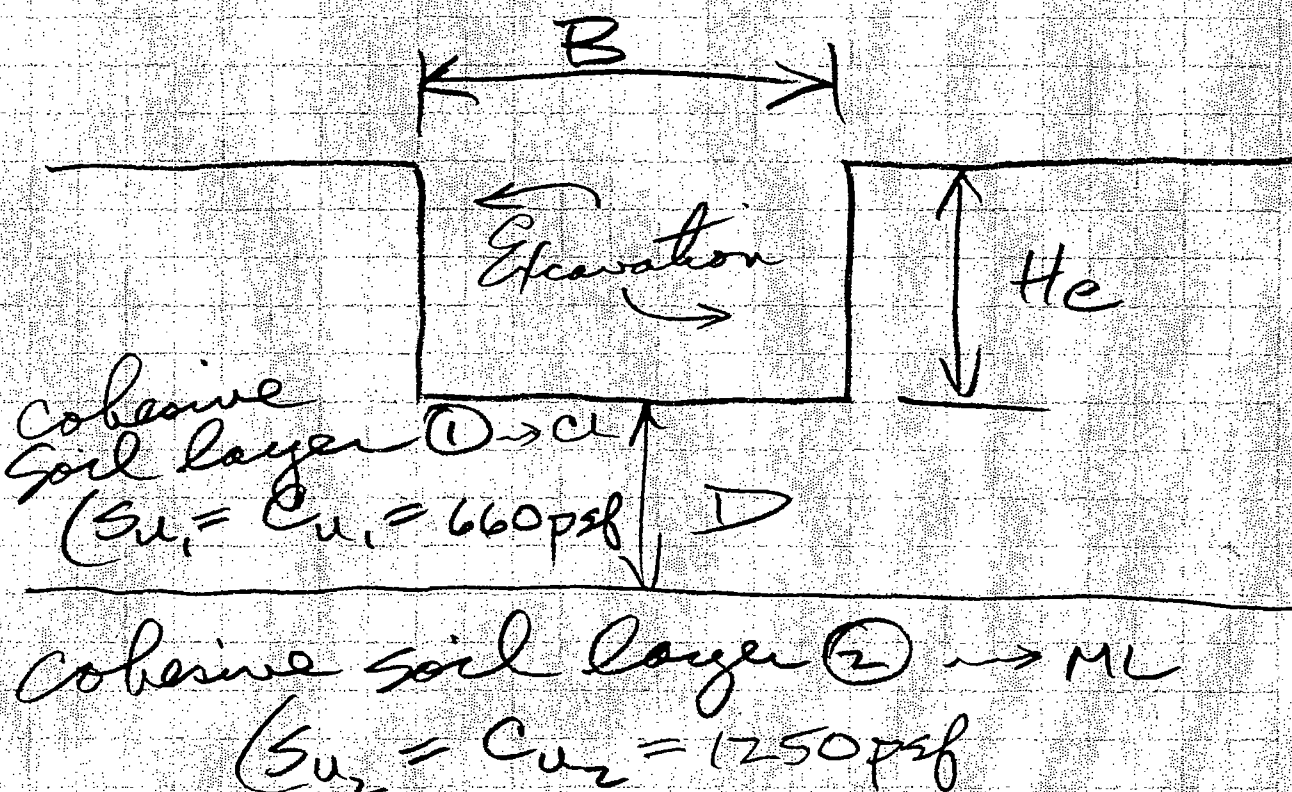
- Bjerrum + Eide's Negative Bearing Capacity (NBC) procedure is based on the following cross sectional model as shown in Fig 5.18 a of the Attachment to this package:



Su_1 can be either > or < Su_2 to use this procedure.

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Governing equation is eqn 5.13 of Attachment to this package:

$$F_b = FS = \frac{(Su_1)(N_{cs})(f_d)(f_s)}{\gamma_T H_e} \rightarrow \frac{psf}{psf}$$

where $N_{cs} \rightarrow$ from Figure 5.18a

$Su_1 = Su(CL) = 660$ psf

$f_d \rightarrow$ from Figure 5.18c $= f(He/B)$

$f_s = 1 + .2\frac{B}{L} \rightarrow$ eqn 5.14

Based on the "Conceptual Design Subsurface Profile" presented as pg 42 of Appendix A, the site specific subsurface cross section relevant to this analysis is:

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WESTON SOLUTIONS

CLIENT/SUBJECT ______ SHEET 7 of ____

TASK DESCRIPTION artesian head in aquifer W.O. NO. ______

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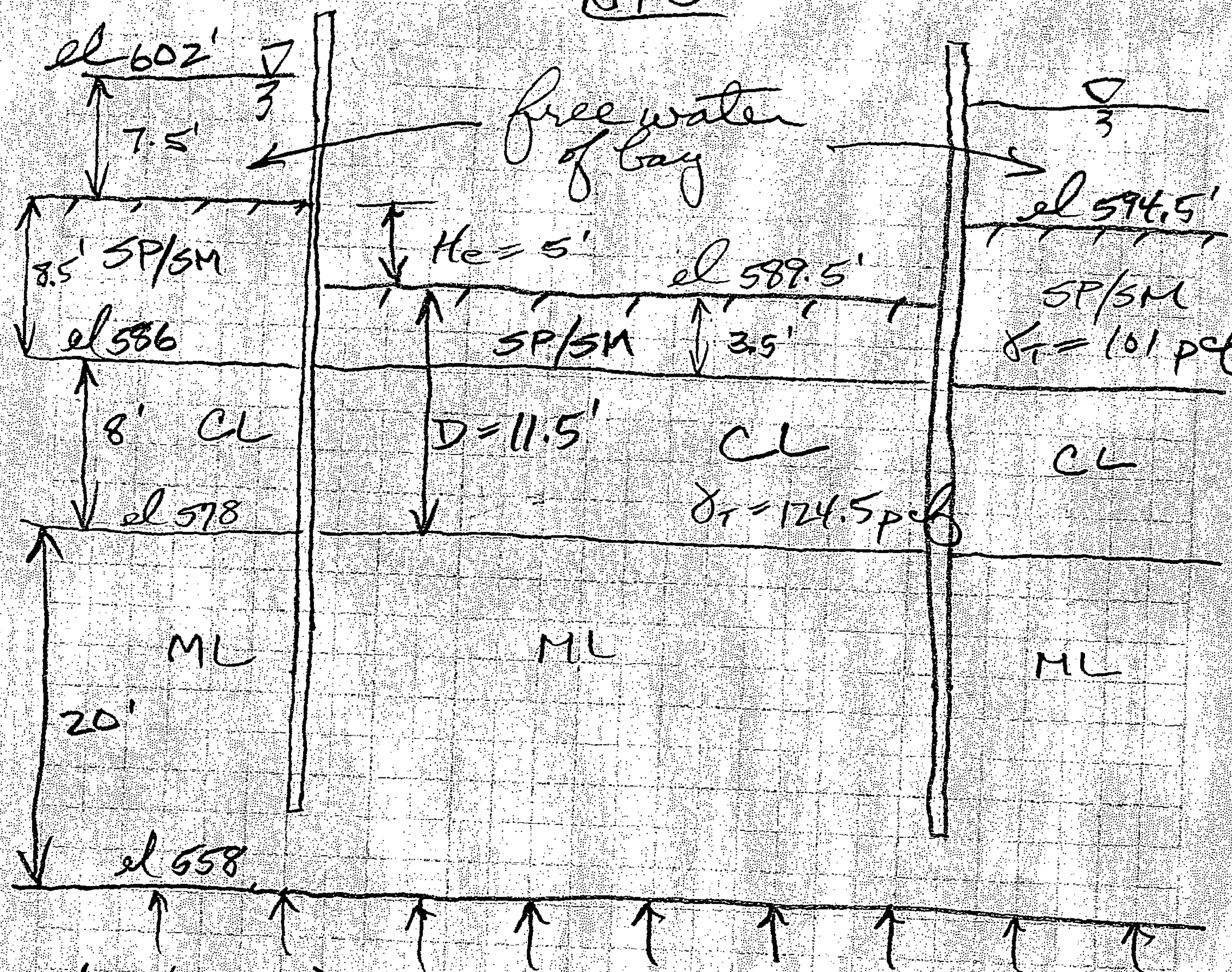
el 617.1 $\overline{\nabla}$ 3

NTS



P_u (ARTESIAN) SP/SM/SC Aquifer

$= (el\ 617.1 - el\ 558)\ \gamma_w = 3688$ psf

Comparing the above Figure with that on the previous page, the following simplifying assumptions were made to complete the

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analysis:

1.) the bay bottom surficial stratum (SP/SM) of 3.5' thickness beneath the base of the excavation was assumed to be part of the underlying CL stratum of cohesive shear strength S_{u_1} $= C_{u_1} = 660$ psf for the analyses

2.) the weight of the 7.5' of bay free water was assumed to act as a surcharge load on the surface of the bay bottom sediments at elev 594.5'. It's magnitude is given by:

$$q\,(\text{free water}) = q_{FW} = 7.5'\,\gamma_w$$
$$= 468 \text{ psf}$$

It's negative contribution to

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the stability of the excavation bottom will be incorporated by adding this surcharge to the denominator of the governing NBC equation presented on page 6.

3) The vertical upward pressure of 3688 psf caused by artesian water pressure in the underlying aquifer also has a negative contribution to the stability of the excavation bottom. It will also be incorporated into the analysis by adding this surcharge to the denominator of the governing NBC equation presented on page 6.

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the modified NBC equation is then:

$$FS = \frac{(Su_1)(N_{cs})(\zeta_D)(\zeta_s)}{\gamma_T He + 468\,psf + 3688\,psf}$$

& $Su_1 = S_u(CL) = 660$ psf

N_{cs}: $Su_2/Su_1 = \frac{1250\,psf}{660\,psf} = 1.89$

& $D = el\ 589.5 - el\ 578 = 11.5'$

$B = 150'$ (see App C)

$\therefore D/B = 11.5'/150' = .077 \leadsto$ use .1

& From Fig 5.18a $\leadsto N_{cs} \approx 10$

For BXL = 150'x200' sheeted dry excav cell (see App C) $\leftarrow \zeta_s = 1 + .2\left(\frac{B}{L}\right) = 1 + .2\left(\frac{150'}{200'}\right) = 1.15$

ζ_H: Figure 5.18C

where $He/B = 5'/150' = .033$

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of $\zeta_d = 1.0+ \rightarrow$ use 1.0

Also $H_e \gamma_t$ for SM/SP excav soil

$= (15')(101 \text{ pcf}) = \quad \text{psf}$

$$\therefore FS = \frac{(660 \text{ psf})(10)(1.0)(1.15)}{505 \text{ psf} + 468 \text{ psf} + 3688 \text{ psf}}$$

$$= \frac{7590 \text{ psf}}{4661 \text{ psf}}$$

$= 1.63 > 1.2$

OK

See pg 141 of Attachment to this package

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WESTON SOLUTIONS

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ATTACHMENT

A-1

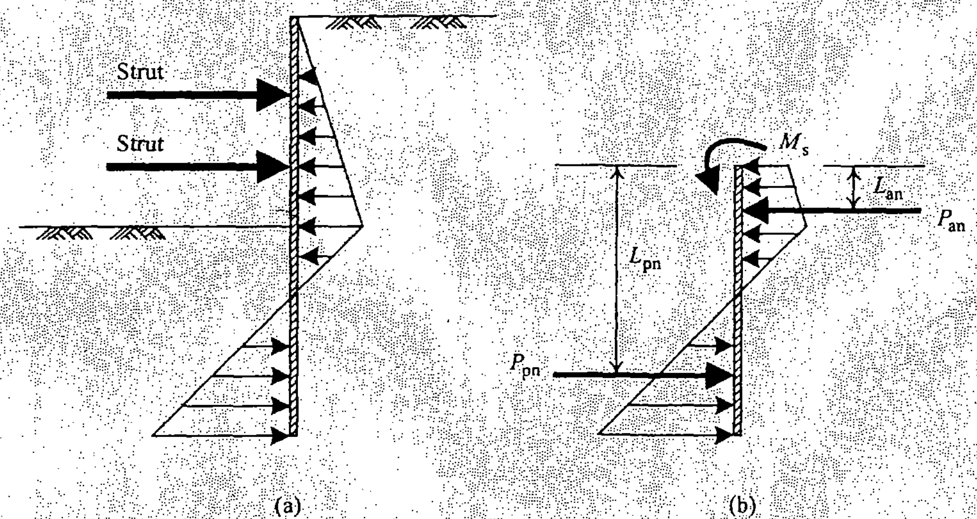

Figure 5.9 Analysis of push-in by the net pressure method: (a) distribution of net earth pressure and (b) force equilibrium of the retaining wall as a free body.

The method is the dimension factor method, as discussed in Section 5.2. According to Burland and Potts' study (1981), the factor of safety obtained following the dimension factor method does not conform to the definition of the factor of safety and may lead to unreasonable results.

Some prefer to reduce the passive earth pressure by a factor of safety. For example, $K_{p,design} = K_p/F_p$. Then, compute the penetration depth by way of the horizontal force equilibrium and the moment equilibrium. This is another way of following the strength factor method (as discussed in Section 5.2). Though the method is logical, when applied to practical design, appropriate factors of safety should be carefully determined.

5.5.2 *Basal heave*

The analyses of the basal heave failure are only applicable to clayey soils and the reasons will be given at the end of this section.

Since $\phi = 0$ for clay, the failure surfaces of bearing capacity failures in clay (e.g. the slope stability problems, the ultimate bearing capacity problems of foundations, etc.) are circular arc surfaces. The basal heave failure due to excavation is also a kind of bearing capacity failure and might also have a main circular arc failure surface. The analysis method for basal heave varies with the assumed shapes of failure surfaces near the ground or excavation surface, though the main failure surface is still a circular arc. As discussed in Section 5.3, the analysis method for basal heave assumes many possible failure surfaces and finds their corresponding factors of safety according to mechanics. The one with the smallest factor of safety is the most likely potential failure surface. Many analysis methods have been proposed for basal heave, the most commonly applied of which are Terzaghi's method, Bjerrum and Eide's method, and the slip circle method. This section will categorize these methods into the bearing capacity method, the negative bearing capacity method, and the slip circle method according to their characteristics.

A-2

5.5.2.1 Bearing capacity method

As shown in Figure 5.10, the soil weight above the level of the excavation surface (plane **abc**) can be seen as the load to cause excavation failure. Supposing a trial failure surface caused by the soil weight within the width of B_1 acts on plane **abc** as is shown in Figure 5.10a, we can find the ultimate load for the width of B_1 following Terzaghi's bearing capacity method with the shear strength along side **bd** considered. The ratio of the ultimate load to the weight of soil within the width of B_1 is the factor of safety for the trial failure surface. Then increase the value of B_1 (which denotes increasing of the size of trial failure surfaces) and find the corresponding factor of safety accordingly until the trial failure surface covers the whole excavation (i.e. $B_1 = B/\sqrt{2}$), as shown in Figures 5.10b and 5.10c. Since the weight of B_1-wide soil on each side of the excavation zone may produce failures, the schematic diagram to calculate the factor of safety will be as illustrated in Figure 5.10d. Following the principle of virtual work, the factor of safety induced from Figure 5.10c and that from Figure 5.10d would be identical. The factor of safety against basal heave (F_b) for the excavation is the smallest one among the safety factors corresponding to the trial failure surfaces.

Figure 5.11 is the profile of a hypothetical excavation case, where the undrained shear strength (s_u) is constant. Following the bearing capacity method, we can obtain the relationship between trial failure surfaces (represented by X/H_e) and their corresponding factors of safety, as shown in Figure 5.12. From the figure we can see that the factor of safety falls with the rising of X (i.e. the expansion of the trial failure surfaces). When X rises to two times the

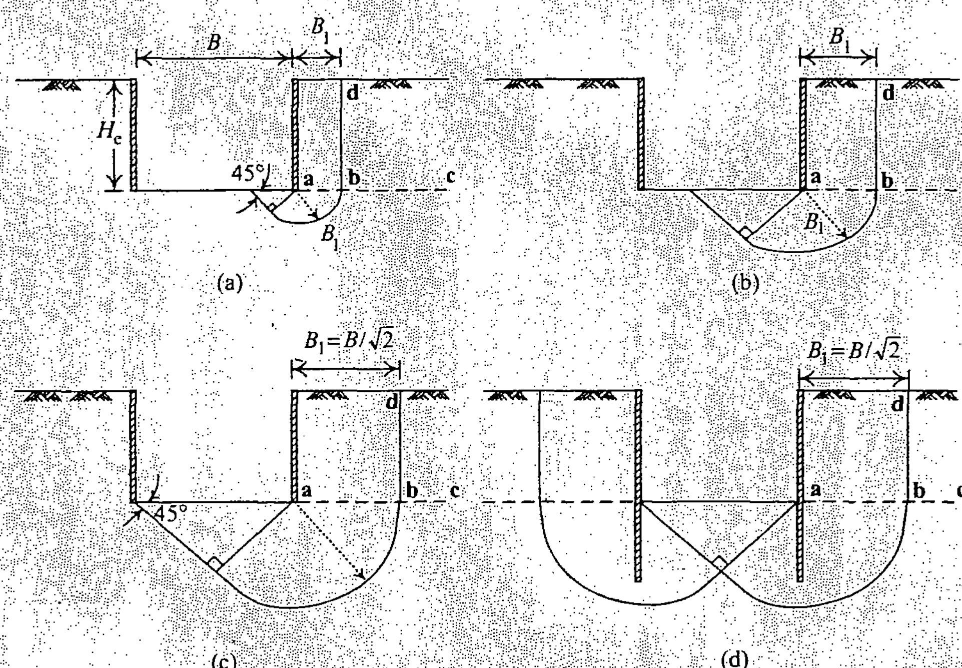

Figure 5.10 Analysis of basal heave by bearing capacity method: (a) a B_1 wide trial failure surface, (b) a second B_1 wide trial failure surface, (c) a third B_1 wide trial failure surface, and (d) both sides of the excavation produce failure surfaces.

A-3

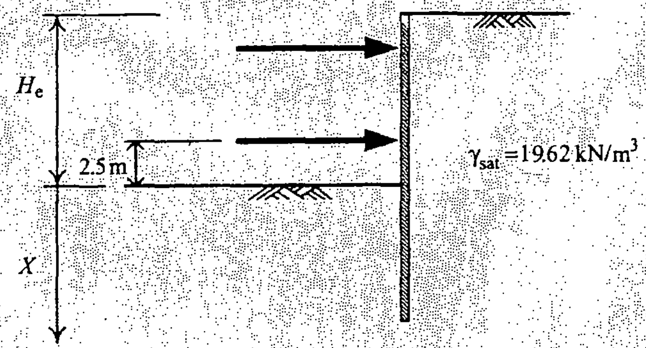

Figure 5.11 Excavation profile of the assumed excavation case.

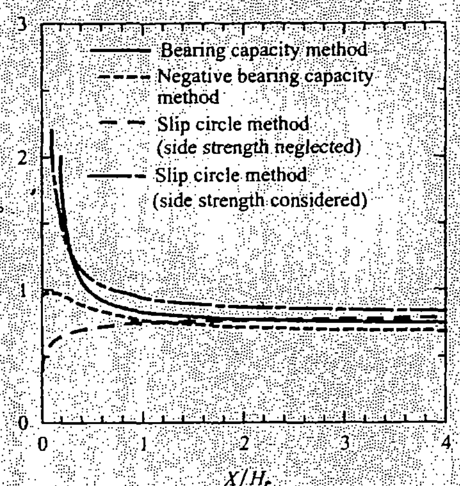

Figure 5.12 Relations between failure circle sizes and factors of safety against basal heave obtained by the bearing capacity method, negative bearing capacity method, and the slip circle method (s_u = 25 kN/m^2).

Su IS CONSTANT WITH DEPTH

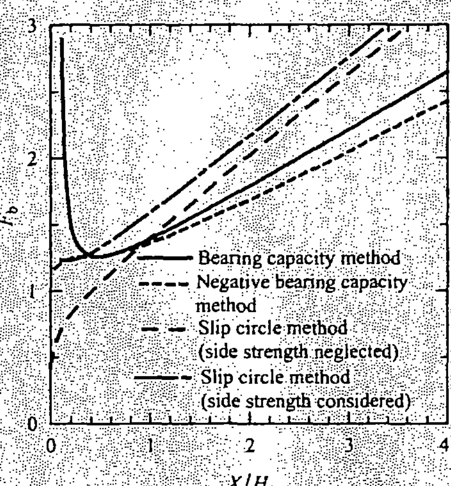

Figure 5.13 Relations between failure circle sizes and factors of safety against basal heave obtained by the bearing capacity method, negative bearing capacity method, and the slip circle method ($s_u/\sigma'_v = 0.3$).

excavation depth, the factor of safety will be approaching a constant. Figure 5.13 is a similar excavation profile with s_u/σ'_v constant. From the figure we can see that F_b also falls with the increase of X. When X comes to a certain value, F_b will be the smallest and then will grow again. It follows that when the trial failure surface covers the whole excavation zone, that is, $X = B/\sqrt{2}$, F_b will not necessarily be its smallest.

A-4

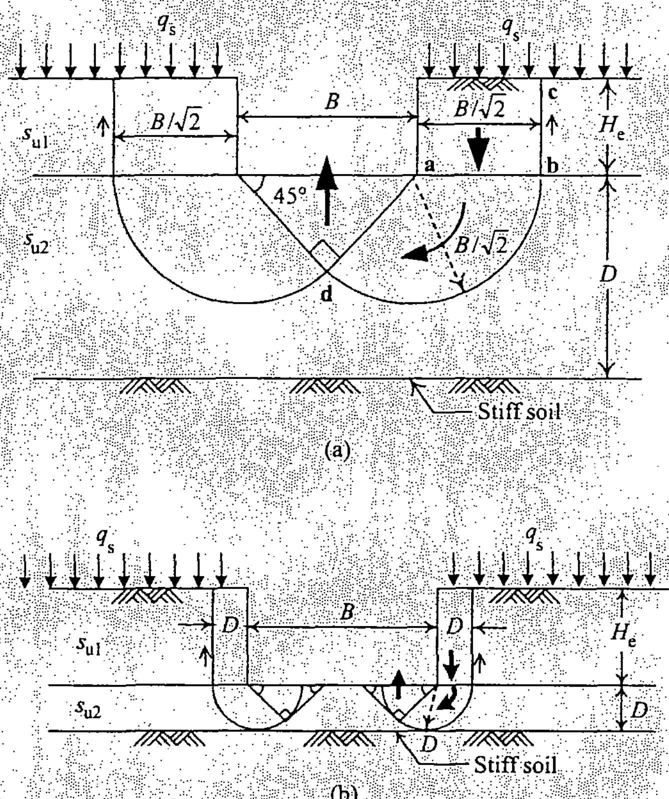

Figure 5.14 Analysis of basal heave using Terzaghi's method: (a) $D \geq B/\sqrt{2}$ and (b) $D < B/\sqrt{2}$.

Terzaghi (1943) did not adopt the above method, where the smallest factor of safety is taken to be the factor of safety against basal heave. Instead, he directly assumed the trial failure surface where $B_1 = B/\sqrt{2}$ (i.e. $X = B/\sqrt{2}$) is the critical failure surface and its corresponding factor of safety is the factor of safety against basal heave, as shown in Figure 5.14. According to Terzaghi's bearing capacity theory, the bearing capacity of saturated clay under plane **ab** can be denoted as $P_{\max} = 5.7s_u$. When the soil weight above plane **ab** is greater than the soil bearing capacity, the excavation will fail. Besides, the failure surface will be restrained by stiff soils. Let D represent the distance between the excavation surface and the stiff soil. We can discuss Terzaghi's method in two parts: $D \geq B/\sqrt{2}$ and $D < B/\sqrt{2}$:

When $D \geq B/\sqrt{2}$

As shown in Figure 5.14a, the formation of a failure surface is not restrained by the stiff soil. Suppose the unit weight of the soil is γ. The soil weight (containing the surcharge q_s) ranges B_1 on plane **ab** will then be:

$$W = (\gamma H_e + q_s)(B_1 \times 1) = (\gamma H_e + q_s)\frac{B}{\sqrt{2}} \tag{5.7}$$

A5

The ultimate load, Q_u, of the saturated clay below plane **ab** will be:

$$Q_u = 5.7s_{u2}(B_1 \times 1) = (5.7s_{u2})\frac{B}{\sqrt{2}} \tag{5.8}$$

When a basal heave failure occurs, vertical plane **bc** can offer shear resistance ($s_{u1}H_e$) and the factor of safety against basal heave (F_b) will be:

$$F_b = \frac{Q_u}{W - s_{u1}H_e} = \frac{5.7s_{u2}B/\sqrt{2}}{(\gamma H_e + q_s)B/\sqrt{2} - s_{u1}H_e} = \frac{1}{H_e} \cdot \frac{5.7s_{u2}}{\gamma + (q_s/H_e) - (s_{u1}/0.7B)} \tag{5.9}$$

where s_{u1} and s_{u2} represent respectively the undrained shear strengths of the soils above and below the excavation surface; q_s denotes surcharge on the ground surface.
When $D < B/\sqrt{2}$

Under such a condition, the failure surface will be restrained by the stiff soil, as shown in Figure 5.14b, and its factor of safety (F_b) will be:

$$F_b = \frac{Q_u}{W - s_{u1}H_e} = \frac{5.7s_{u2}D}{(\gamma H_e + q_s)D - s_{u1}H_e} = \frac{1}{H_e} \cdot \frac{5.7s_{u2}}{\gamma + (q_s/H_e) - (s_{u1}/D)} \tag{5.10}$$

For most excavation cases, Terzaghi's factor of safety (F_b) should be greater than or equal to 1.5 (Mana and Clough, 1981; JSA, 1988).

Assuming that the penetration depth of the retaining wall is deep enough, the failure surface may be formed as illustrated in Figure 5.15a, which is one of the possible failure modes. According to the analysis on the basis of the principle of virtual work, the factor of safety for a failure surface as illustrated in Figure 5.15a is close to that of Eqs 5.9 and 5.10. The only difference is that the failure surface in Figure 5.15a ranges wider (with the extra failure surface **be**) and the average soil strength on the failure surface is higher than that in Figure 5.15b (assuming the undrained shear strength of clay increases with the increase of depth).

As shown in Figure 5.15b, assuming the penetration depth of the retaining wall is not deep enough, the calculation of the factor of safety will still follow Eqs 5.9/5.10. That is to say, the value of the factor of safety against basal heave has nothing to do with the existence of

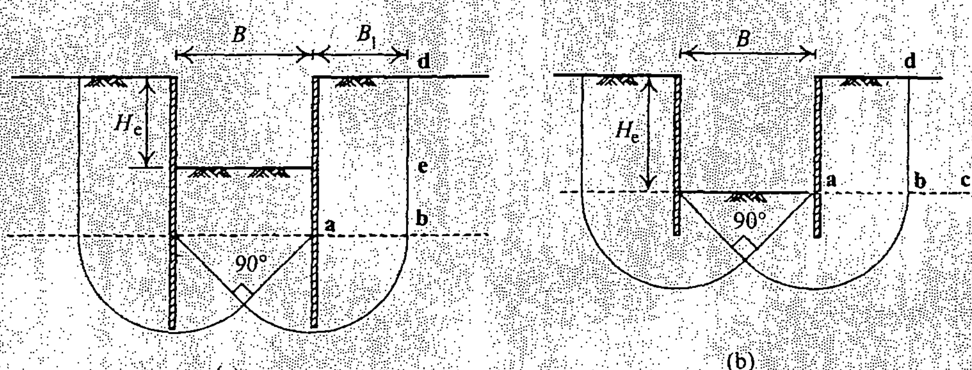

Figure 5.15 Relation between the embedded part of the retaining wall and the failure surface: (a) large penetration depth and (b) small penetration depth.

A-6

the retaining wall according to the equations. However, theoretically speaking, the retaining wall with high stiffness may be capable of restraining basal heave failures. Thus, the actual factor of safety should be greater than the result from Eq. 5.9 or 5.10 though there does not exist a suitable way to estimate it.

The bearing capacity method or Terzaghi's method is suitable for shallow excavations, where the excavation width (B) is larger than the excavation depth (H_e). For deep excavations, $B < H_e$, the bearing capacity method or Terzaghi's method may not yield reasonable results because the method assumes that the failure surface extends up to the ground surface and that the shear strength of clay is fully mobilized all the way to the ground surface, neither of which is necessarily true for deep excavations.

5.5.2.2 Negative bearing capacity method

The *negative bearing capacity method* assumes that the unloading behavior caused by excavation is analogous to the building foundation being subject to an upward loading and that the shape of the failure surface is similar to the failure mode of the deep foundation. Then, using the bearing capacity equation for the deep foundation, we can obtain the ultimate unloading pressure. The factor of safety is the ratio of the ultimate unloading pressure to the unloading pressure. As shown in Figure 5.16, assuming various failure surfaces to analyze (representing different B_1-values) and finding their separate corresponding factors of safety, the smallest factor of safety among them is the factor of safety against basal heave for the excavation.

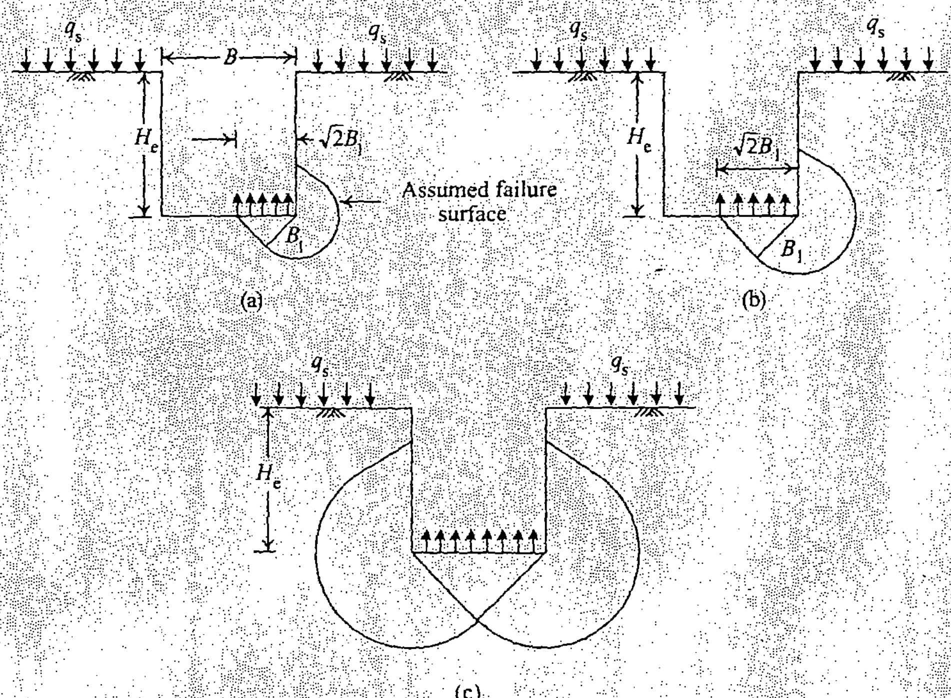

Figure 5.16 Analysis of basal heave failure by negative bearing capacity method: (a) a $\sqrt{2}B_1$ wide failure surface, (b) another $\sqrt{2}B_1$ wide failure surface, and (c) Failure surface covers the whole excavation bottom.

A-7

Figures 5.12 and 5.13 also illustrate the changing tendencies of the factor of safety against basal heave with the size of the trial failure surfaces for s_u = constant or s_u/σ'_v = constant respectively, following the negative bearing capacity method (as for the excavation profile, please see Figure 5.11), where the bearing capacity factor, N_c, can be determined according to Skempton (1951), as shown in Figure 5.17. N_c can also be calculated by the following equation:

$$N_{c(\text{rectangular})} = N_{c(\text{square})}\left(0.84 + 0.16\frac{B}{L}\right) \tag{5.11}$$

As shown in Figures 5.12 and 5.13, we can see that for soils where s_u = constant, the factors of safety against basal heave derived from the negative bearing capacity method will decrease with the increase of the size of the trial failure surfaces (which means X increases). For soils where s_u/σ'_v = constant, the factors of safety against basal heave derived from the negative bearing capacity method will increase with the increase of the size of the trial failure surfaces.

Like Terzaghi's method, Bjerrum and Eide (1956) did not yield the factor of safety against basal heave by finding the smallest, as just mentioned. Instead, they assumed the failure surface where the radius of the circular arc is equal to $B/\sqrt{2}$ is the critical failure surface and the corresponding factor of safety is the one against basal heave (see Figure 5.16c). The factor of safety can be expressed as follows:

$$F_b = \frac{N_c \cdot s_u}{\gamma \cdot H_e + q_s} \tag{5.12}$$

where q_s is the surcharge on the ground surface and N_c is Skempton's bearing capacity factor as shown in Figure 5.17.

Since N_c has taken into account the effects of the embedment depth of foundations and excavation size, Eq. 5.12 is equally valid for shallow and deep excavations, as well as rectangular excavations.

According to Reddy and Srinivasan's study (1967), NAVFAC DM 7.2 (1982) modified Bjerrum and Eide's method to apply the method to the excavations where there are stiff soils below the excavation surfaces or there are two layers of soils. As shown in Figure 5.18, the

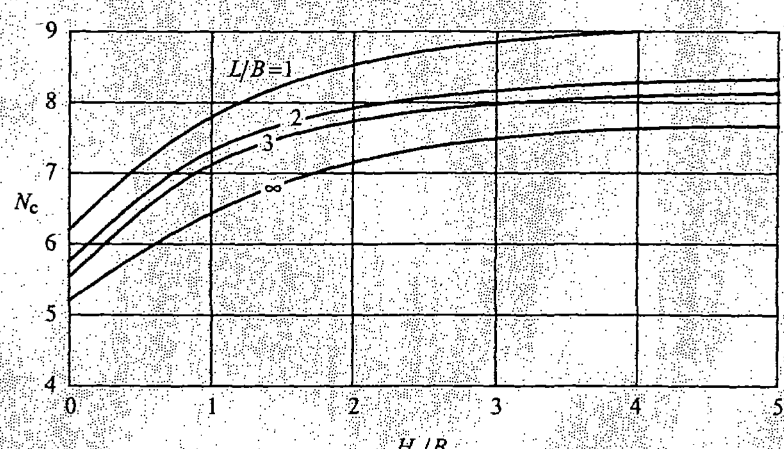

Figure 5.17 Skempton's bearing capacity factor (Skempton, 1951).

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Extended Bjerrum and Eide's method can be expressed as follows:

$$F_b = \frac{s_{u1} N_{c,s} f_d f_s}{\gamma H_e} \tag{5.13}$$

where

γ = unit weight of the soil
H_e = excavation depth
s_{u1} = undrained shear strength of the upper clay
s_{u2} = undrained shear strength of the lower clay
$N_{c,s}$ = bearing capacity factor that does not consider the excavation depth. This can be determined according to Figure 5.18a or 5.18b with the values of D/B (the ratio of the distance from the excavation surface to the lower soils to the excavation width) and s_{u2}/s_{u1} given
f_d = depth correction factor, which can be found in Figure 5.18c
f_s = shape correction factor, which can be estimated by the following equation:

$$f_s = 1 + 0.2\frac{B}{L} \tag{5.14}$$

where B refers to the excavation width and L the excavation length.

Like Terzaghi's method, when there exists stiff soil below the excavation surface, the failure surfaces assumed by Bjerrum and Eide's method and by the Extended Bjerrum and Eide's method would also be restrained by the stiff soil. The stiff soil may be stiff clays, sandy soils or gravel soils. To conduct the stability analysis of the basal heave failure, we can use Eq. 5.13, where $N_{c,s}$ can be found in Figures 5.18a or 5.18b. The latter is a simplified version of the former, assuming that the failure circle will be tangent to the lower soils when s_{u2}/s_{u1} is very large.

If the penetration depth of the retaining wall is deep enough, Bjerrum and Eide's method computes the factor of safety in a way similar to Terzaghi's method. That is to say, the failure surface will be formed in a deeper level, similar to what is illustrated in Figure 5.15a. Under such conditions, Eq. 5.12 is still workable to estimate the factor of safety with the slight differences of average soil strengths on failure surfaces. When the H_p is not large enough, the calculation of the factor of safety will still follow Eqs 5.12/5.13. That is to say, the value of the factor of safety against basal heave has nothing to do with the existence of the retaining wall according to the equations.

The negative bearing capacity method or Bjerrum and Eide's method take into account the effects of excavation shape, width, and depth. Therefore, the methods are applicable to various shapes of excavations, shallow excavations as well as deep excavations.

For most excavations, the factor of safety obtained according to Bjerrum and Eide's method (F_b) should be larger than or equal to 1.2 (JSA, 1988).

5.5.2.3 Slip circle method

Let the trial failure surfaces of the basal heave failure be assumed to be basically circular arcs, and separately compute the ratios of the resistant moments to the driving moments for the trial circular arc failure surfaces. The smallest factor of safety among them is then the factor of safety against basal heave for the excavation. The method is designated as the slip circle

A-9

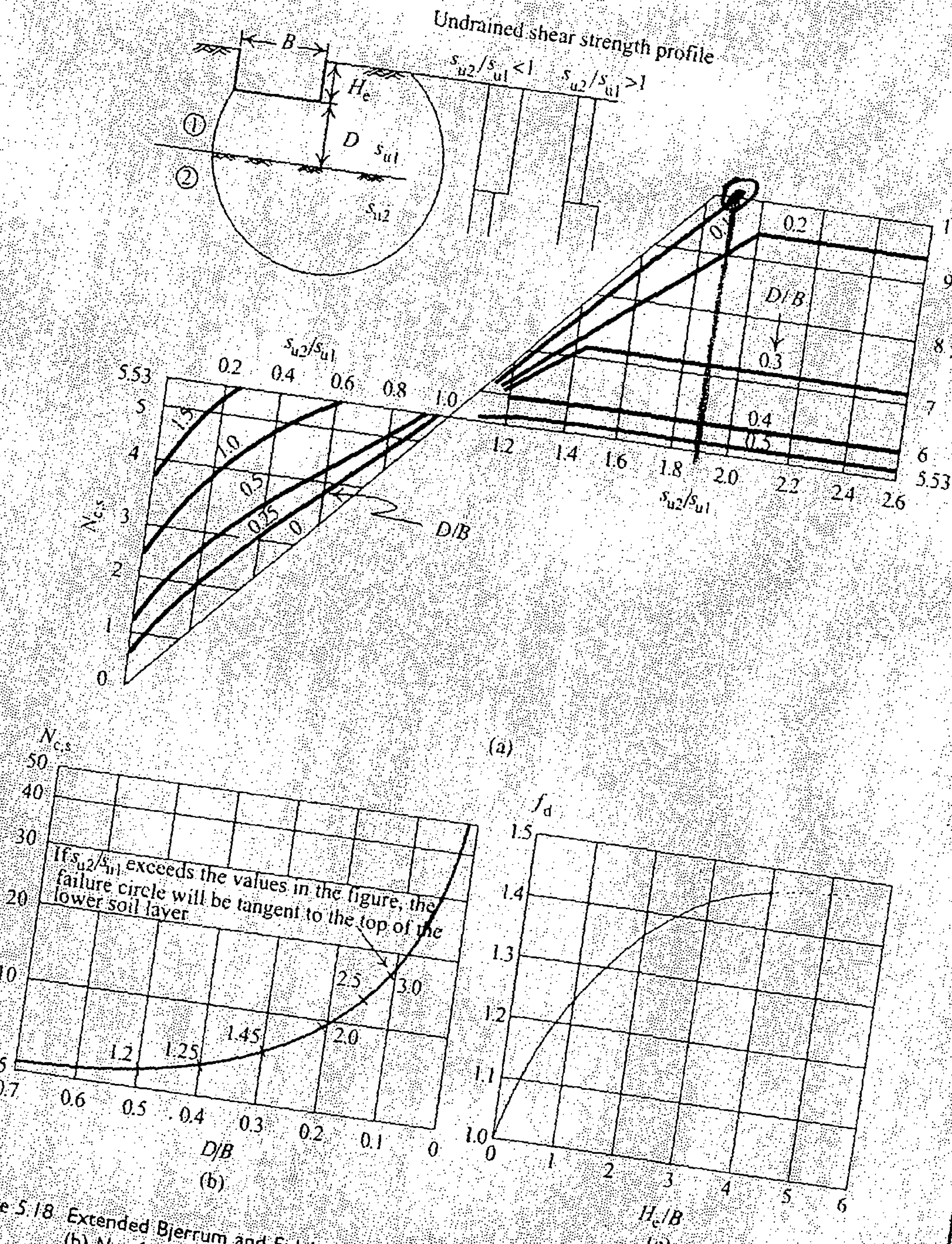

Figure 5.18 Extended Bjerrum and Eide's method: (a) $N_{c,s}$ for failure circles passing two soil layers, (b) $N_{c,s}$ for failure circles tangent to the top of the lower soil layer, and (c) f_d modified by the width (NAVFAC DM7.2, 1982; Reddy and Srinivasan, 1967).

APPENDIX E
EXIT GRADIENT ANALYSIS

WESTON SOLUTIONS

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CLIENT/SUBJECT Ashland, Wi Superfund Site

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TASK NO.

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EXIT GRADIENT ANALYSIS

- Vertically upward hydraulic exit gradients within a dry excavation cell footprint could ~~potentially~~ destabilize the excavation bottom soils (i.e. SP/SM predominantly granular materials) if sufficiently high. The nature of this process would be a fluidization of these materials similar to liquefaction. This phenomena is referred to as "piping" in the Geotechnical literature. If piping manifests itself at the surface of the fluidized cohesionless soil stratum, the physical process is referred to as "sand boiling".

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- Two mechanisms, one natural + one man-induced, could, on their own or in combination, result in piping in the excavation bottom SP/SM surficial soils. These include

1.) The upward gradient which is naturally present in the SP/SM, CL + ML soil layers beneath the excavation bottom induced by the artesian head in the underlying granular soil aquifer (SP/SM/SC soils).

2.) The upward gradient which will be induced immediately adjacent to the sheet pile wall following its installation + dewatering of the free water inside a given sheet piling cell from free water in the bay (el. 602' surface outside of the cell.

- The first mechanism was evaluated & quantified using computer software package SEEP W, a windows based seepage analyses program. The input data to SEEP W was based on the following:

a) The Conceptual Design Subsurface Profile developed in Appendix A as summarized on page 42 of this package. This Figure is reproduced as page 4 of this package.

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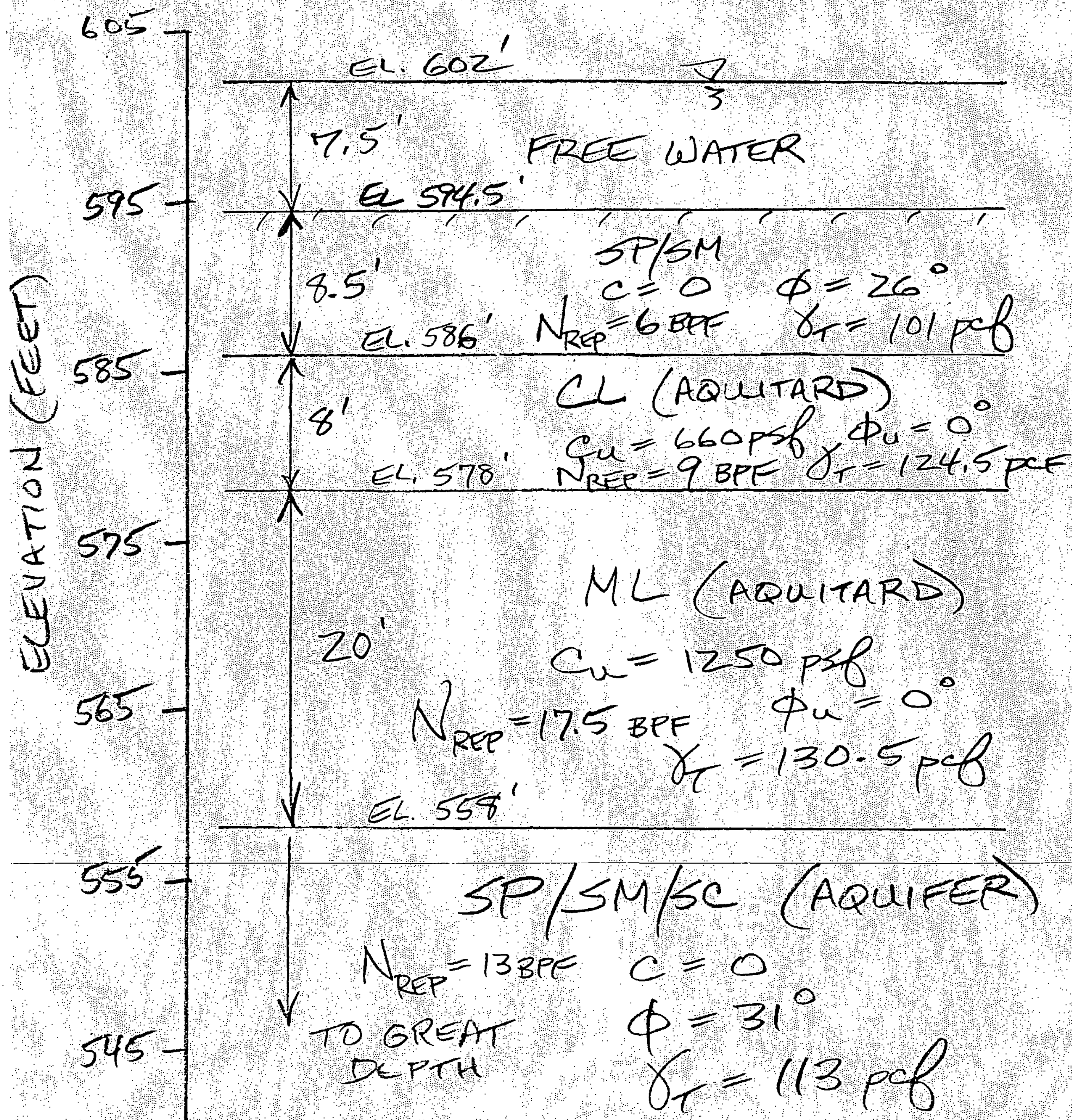

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b.) An excavation width (B) dimension of 150'.

c.) the following assumed values of vertical permeability (K_v) for the 4 relevant soil layers:

SP/SM (Excavation Bottom): 1×10^{-2} cm/s

CL (Aquiclude): 1×10^{-7} cm/sec

ML (Aquiclude): 1×10^{-5} cm/sec

SP/SM/SC (Artesian Aquifer): 1×10^{-3} cm/sec

d.) An artesian hydraulic head was applied in the program input file based on an elevation of 617.1'. This resulted in vertically upward flow in all 4 soil layers as is evident from the ~~SEEP~~ W output presented subsequently.

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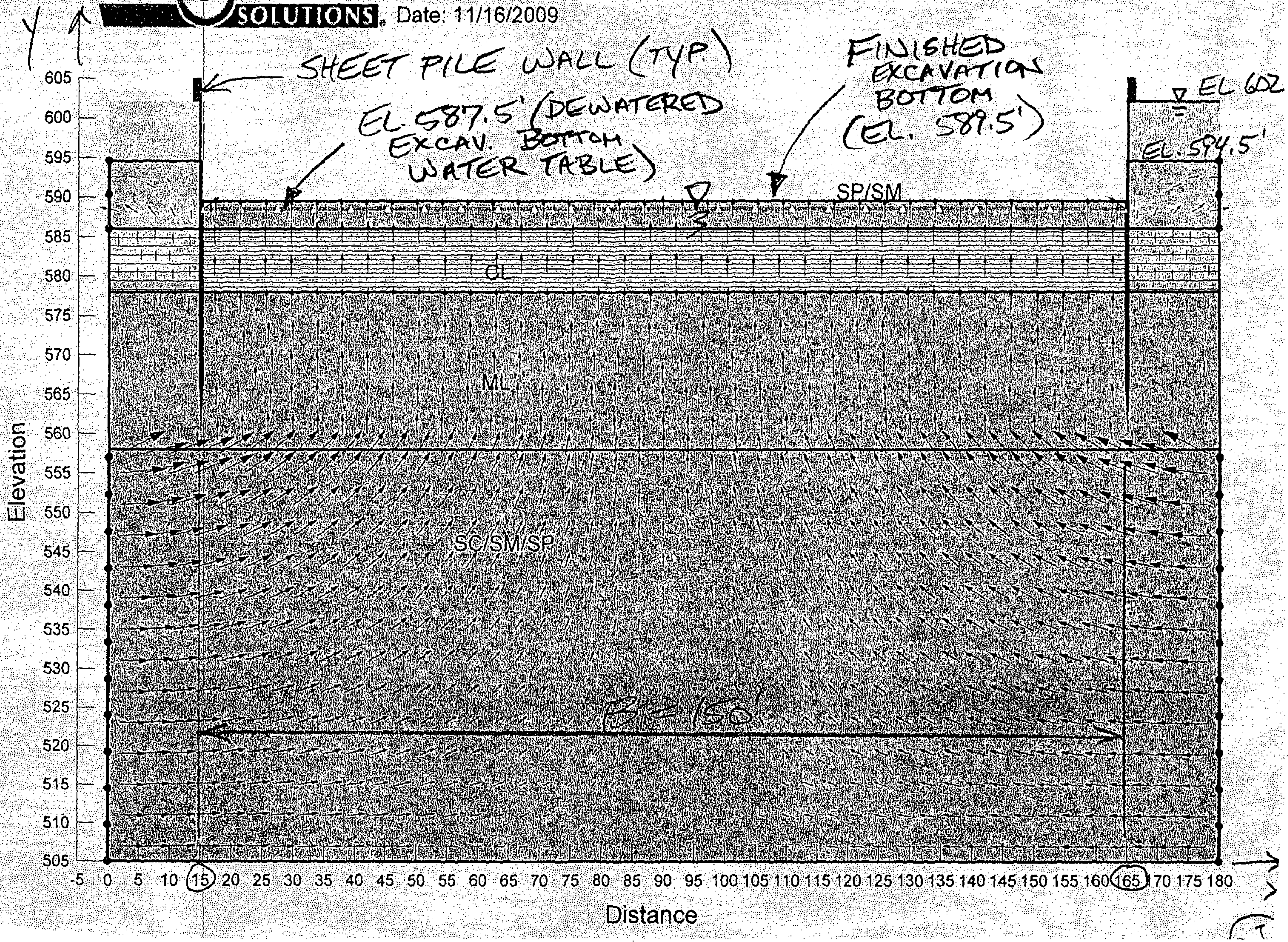
d.) Sheet piling retaining structures spaced at 150' apart with top & bottom elevations of 605' & 560' (see Appendix B) were modelled in SEEP W to reduce underflow beneath the sheets.

e.) the finished excavation bottom grade (el 589.5') & a reduced water table elevation which is 2' lower than the excavation bottom grade (ie. el 587.5') achieved via a system of dewatering trenches or equivalent was assumed in the SEEP W analysis.

Graphical results from SEEP W are presented on page 7. the upward

Project: Ashland Superfund
File Name: Water Water 150.gsz
Name: Water-Water Steady-State Face
Date: 11/16/2009



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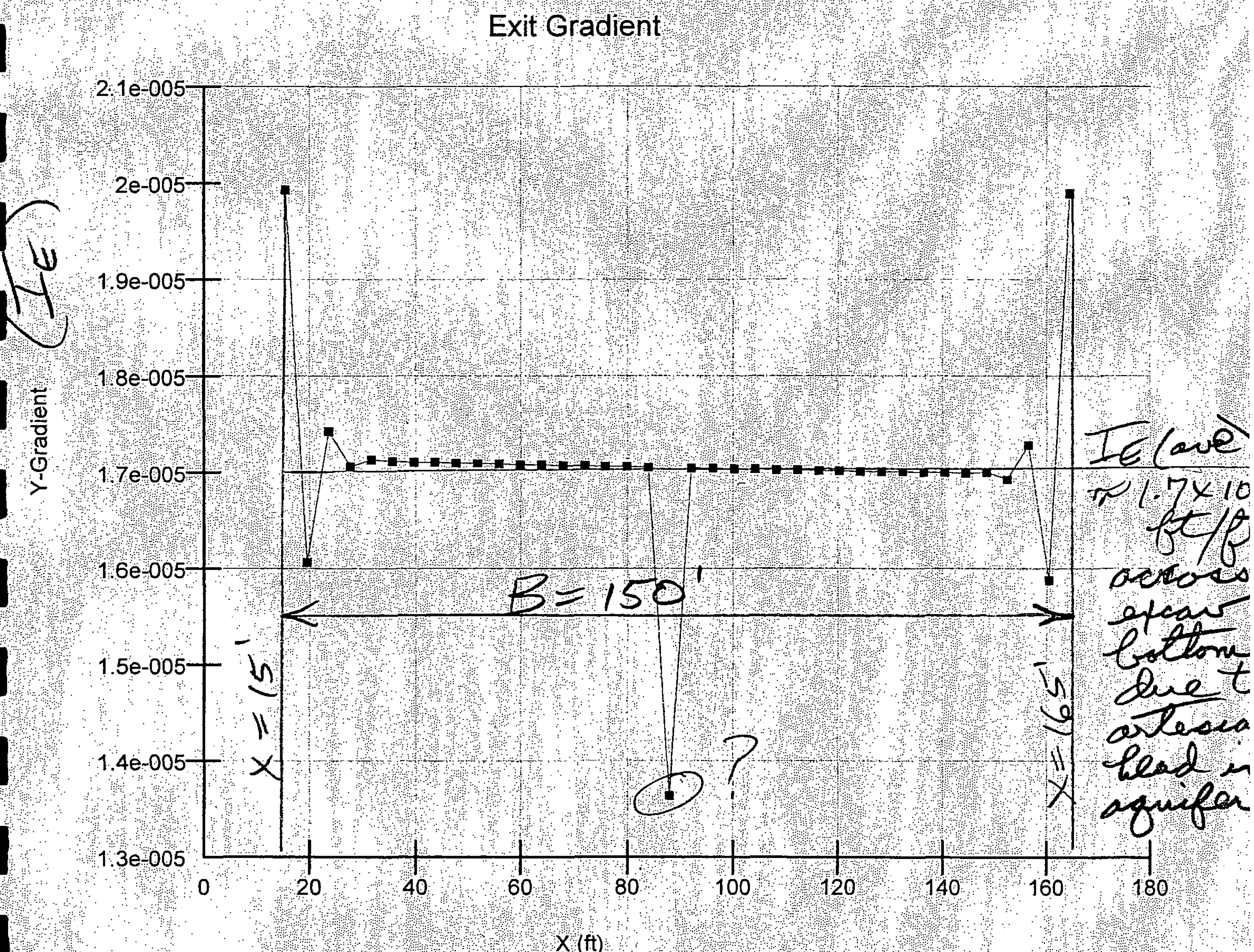
flow created by the 2 mechanisms discussed above is self evident from an inspection of this Figure. In addition, a second output printout from SEEP W presented on page 9 depicts the upward vertical exit gradient (Ie) (4 of is) across the width of the excavation. These values of Ie are relevant at elevation 587.5', the assumed dewatered groundwater elevation below the excavation bottom. Note that the average Ie value across the excavation bottom is approx. 1.7×10^{-5} ft/ft, an insignificant value, which is assuredly due to the very significant head loss/water pressure loss induced in the upward


Exit Gradient
2.1e-005
2e-005
1.9e-005
1.8e-005
1.7e-005
1.6e-005
1.5e-005
1.4e-005
1.3e-005
Y-Gradient
0
20
40
60
80
100
120
140
160
180
X (ft)
(IE)
B = 150'
X = 15'
X = 165'
?
IE (ave)
≈ 1.7×10
ft/f
across
excav
bottom
due t
artesia
head i
aquifer

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flow as it migrates through the 28 aquitard thickness. Note also that the highest exit gradient predicted by SEEP W occurs just inside of the sheet piling walls. This is consistent with known field behavior + theory as noted by Harr (see pgs 11 & 12).

- Even though SEEP W permits estimation of $I_{E\,max}$ immediately adjacent + inside of the sheet piling walls ($i_E \approx 2 \times 10^{-5}$ ft/ft (±)), a more conservative estimate of $I_{E\,max}$ can be completed by hand using a procedure by Harr presented on pages 11 through 14 of this package.

(11)

Once again it is advisable to use an indirect approach (assuming t and finding the corresponding z) for the determination of the pressure distribution along the contour of the structure (see Example 5-2).

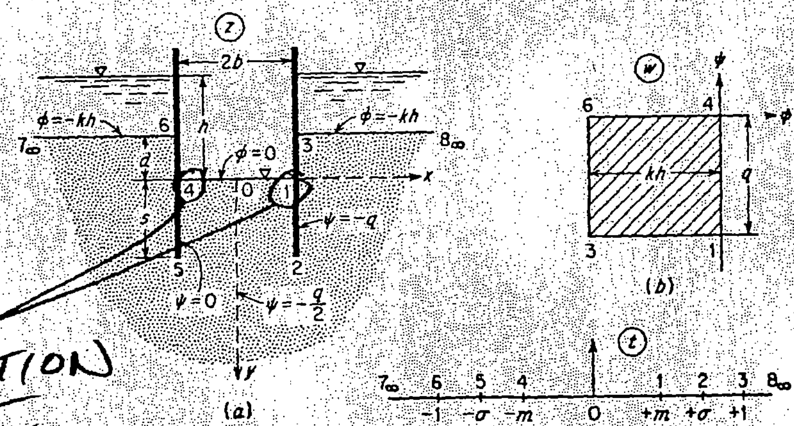
For the exit gradient (point 3), using Eq. (2), Sec. 5-1, we readily obtain

$$I_E = \frac{hm'}{2b\left(K' - E' + \frac{d}{b}E\right)} \tag{14}$$

where the modulus is as is given in Fig. 5-33.

5-9. Double-wall Sheetpile Cofferdam

Figure 5-34 represents a section through a double-wall cofferdam consisting of two rows of sheetpiles. After the sheetpiles are driven, the



LOCATION OF $I_{E_{max}}$

Fig. 5-34

soil between them is excavated to a depth d below the ground surface. We seek in this problem to determine the discharge quantity and the factor of safety with respect to piping.

Noting in Fig. 5-34 that the z plane and t plane are precisely the same as in Sec. 5-8, we have immediately for the required transformation between them [Eq. (12), Sec. 5-8],

$$z = \frac{2b}{\pi}\left[\left(E' - K' + \frac{d}{b}E\right)u + \left(K' + \frac{d}{b}K\right)E(u)\right] \tag{1}$$

where sn $u = t/m$, and the modulus m can be obtained directly from Fig. 5-33.

Ref: "Groundwater & Seepage", M. E. Harr

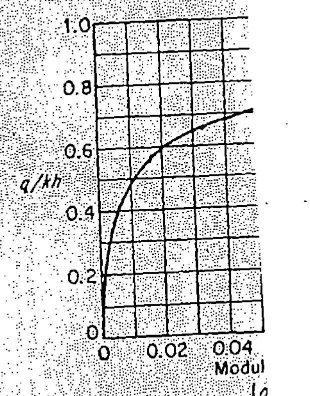
It is convenient ir
5-34b. Hence, for tl

$$w = \frac{M}{m}\int$$

where, as above, sn
Considering the c
hence sn $u = 1$, $u =$

At points 3, $t = 1$ an
and

A plot of Eq. (4) as
Fig. 5-35.



Recalling that $q =$
the direction of flow,
bottom of the excava

For the determinatio
the excavation (at po
obtain the relation

$$I_E = I_1$$

12

It is convenient in this problem to take the w plane as shown in Fig. 5-34b. Hence, for the mapping of the w plane onto the t plane, we have

$$w = \frac{M}{m}\int_0^t \frac{dt}{\sqrt{(1-t^2)(1-t^2/m^2)}} - \frac{iq}{2} = Mu - \frac{iq}{2} \tag{2}$$

where, as above, sn $u = t/m$.

Considering the correspondence at points 1, $t = m$ and $w = -iq$; hence sn $u = 1$, $u = K$ and

$$M = -\frac{iq}{2K} \tag{3}$$

At points 3, $t = 1$ and $w = -kh - iq$; hence sn $u = 1/m$, $u = K + iK'$, and

$$q = \frac{2khK}{K'} \tag{4}$$

A plot of Eq. (4) as a function of the modulus (Fig. 5-33) is given in Fig. 5-35.

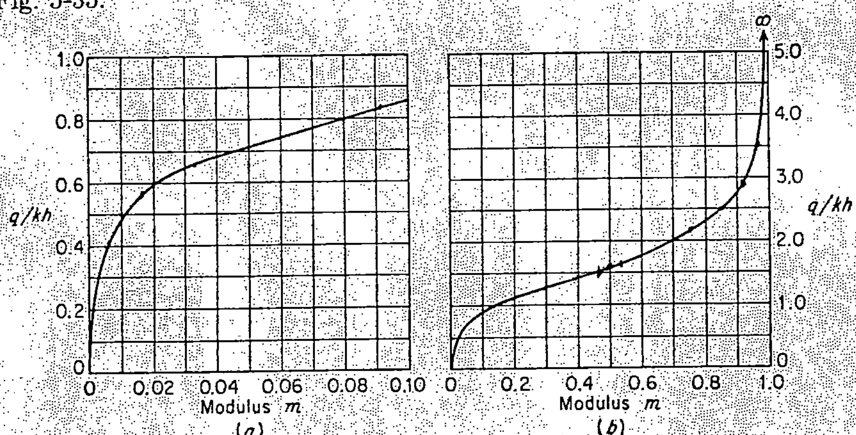


FIG. 5-35

Recalling that $q = kIA$, where A is the area of the section normal to the direction of flow, we find, for the average exit gradient along the bottom of the excavation,

$$I_{av} = \frac{q}{2kb} \tag{5}$$

For the determination of the maximum exit gradient along the base of the excavation (at points 1 and 4, $t = \pm m$), from Eq. (2), Sec. 5-1, we obtain the relation

$$I_E = I_1 = I_4 = \frac{h\pi}{2bK'[K' + (d/b)K](m^2 - \sigma^2)} \tag{6}$$

13

where σ^2 is defined by Eq. (8), Sec. 5-8. A plot of Eq. (6) in terms of $I_E s/h$ is given in Fig. 5-36.

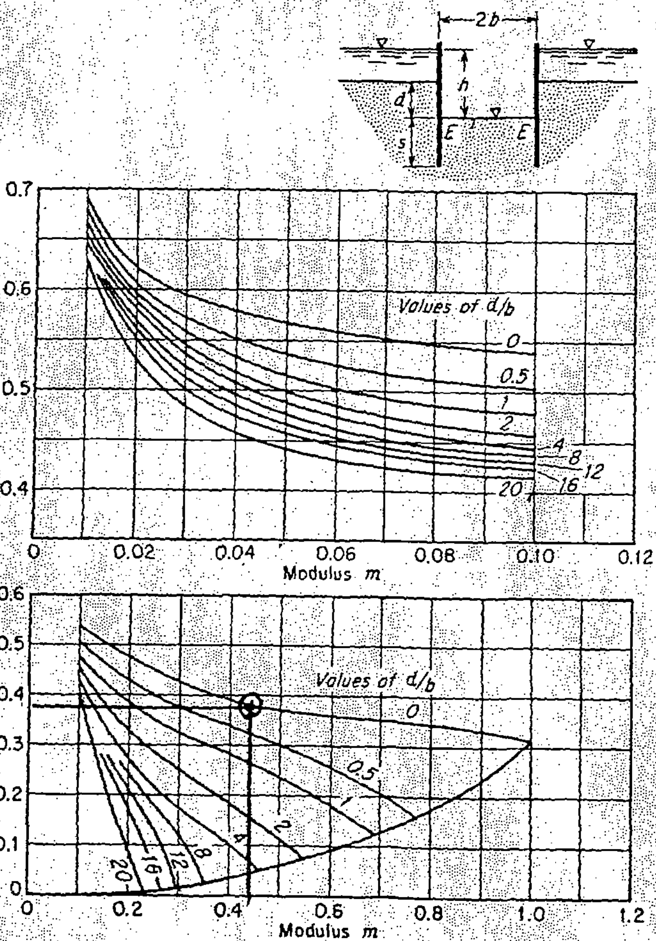


Fig. 5-36

Example 5-3. In Fig. 5-34, $h = 10$ ft, $d = 4$ ft, $2b = 40$ ft, and $s = 10$ ft. Determine (*a*) the reduced quantity of flow (q/k), (*b*) the average exit gradient, and (*c*) the maximum exit gradient.

From Fig. 5-33, with $s/b = 0.5$ and $d/b = 0.2$, we obtain the modulus $m = 0.35$. Then, from Fig. 5-35, we find $q/kh = 1.3$ and hence $q/k = 13$ ft.

From Eq. (5), we obtain the average gradient $I_{av} = 13/40 = 0.32$.

Next, entering Fig. 5-36 with $m = 0.35$ and $d/b = 0.2$, we find $I_E s/h = 0.39$, whence $I_E = 0.39$. Thus the factor of safety with respect to piping will be $1/0.39 \approx 2.6$.

PROBLEMS

1. Show that the transformation Eq. (3), Sec. 5-2, is valid for the points A and G of Fig. 5-2*a*.

2. Obtain the
3. Obtain the
surface (of infini
4. Demonstrat
from Eq. (16), S
5. Verify that
given by Eq. (1)
6. Show that
sheetpile.
7. Derive the
discuss the natur
½, and (c) great
8. Noting that
points A and D
into $t = \cos(\pi w$
9. A 20 ft wid
The head loss is
piping along the
of the structure.
10. Verify Eqs
11. Verify Eqs

12. For the se
obtain the gener
quantity of seep
and (*c*) the press
piling.

13. Solve Prob
Prob. 3.

14. For the se
estimate the fact
(*a*) uplift force,
moment due to
($\gamma_m = 124.8$ pcf)

(8)

ubstituting $(\sigma^2 - 1)$

$KK' = \pi/2$, yields

(9)

$n',v)$ (10)

ion of the modulus
be plotted to yield
Such a plot was
5-33. We note in

(11)

7 [Eq. (4)] for the

we obtain for the

$E(u)\Big]$ (12)

t plane, we have

$^{1} t \leq \pi$ (13)

(14)

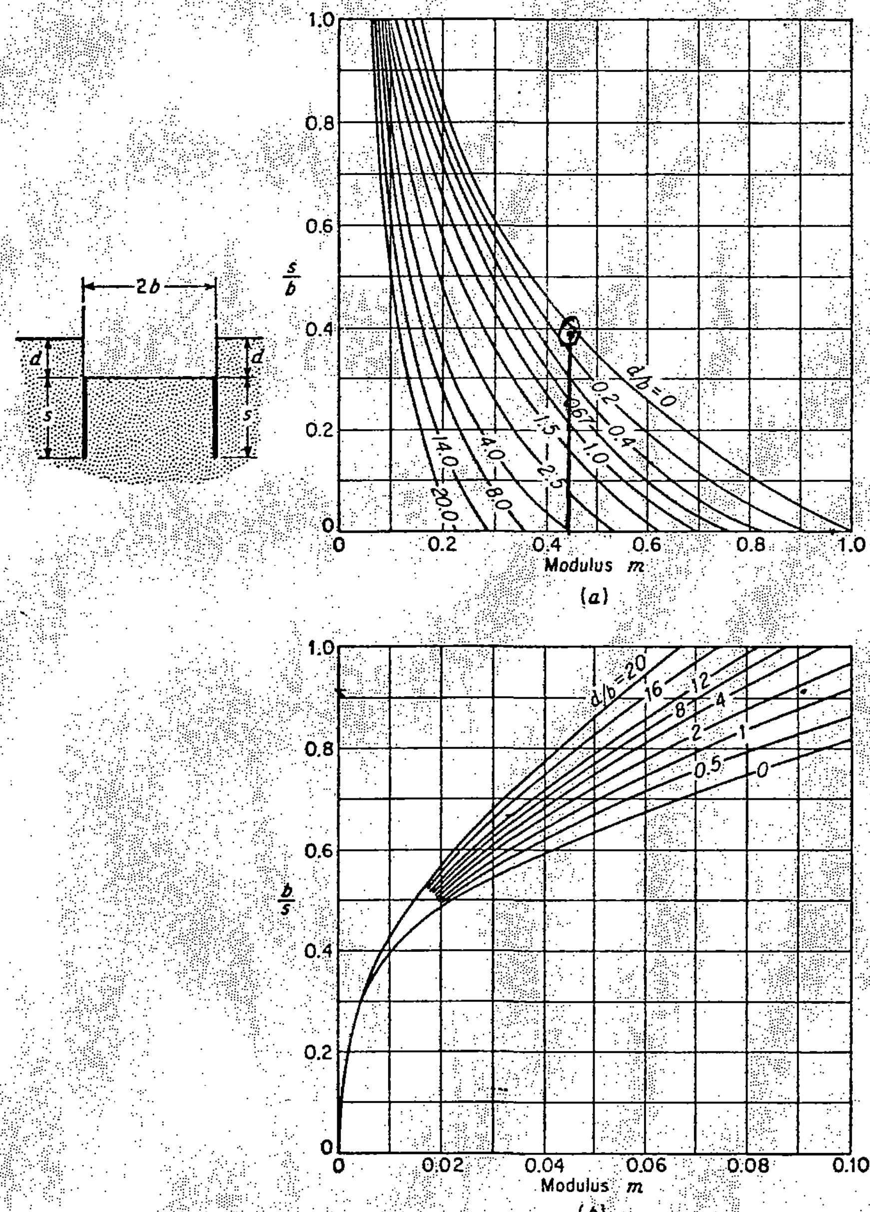


Fig. 5-33

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this analysis would be consistent with the very conservative assumption that the entire subsurface environment consists of granular soils (ie. the ML & CL aquicludes are assumed to not be present), and therefore much lower head loss/water pressure loss occurs in the underflow beneath the sheet piling before the underflow emerges just inside the sheet pile walls at points 1 + 4 of Fig 5-34 (see pg 11) with max exit gradient I_{Emax}.

- the Harr analysis is as follows:
  Based on Fig 5-34a of Harr (see pg 11):

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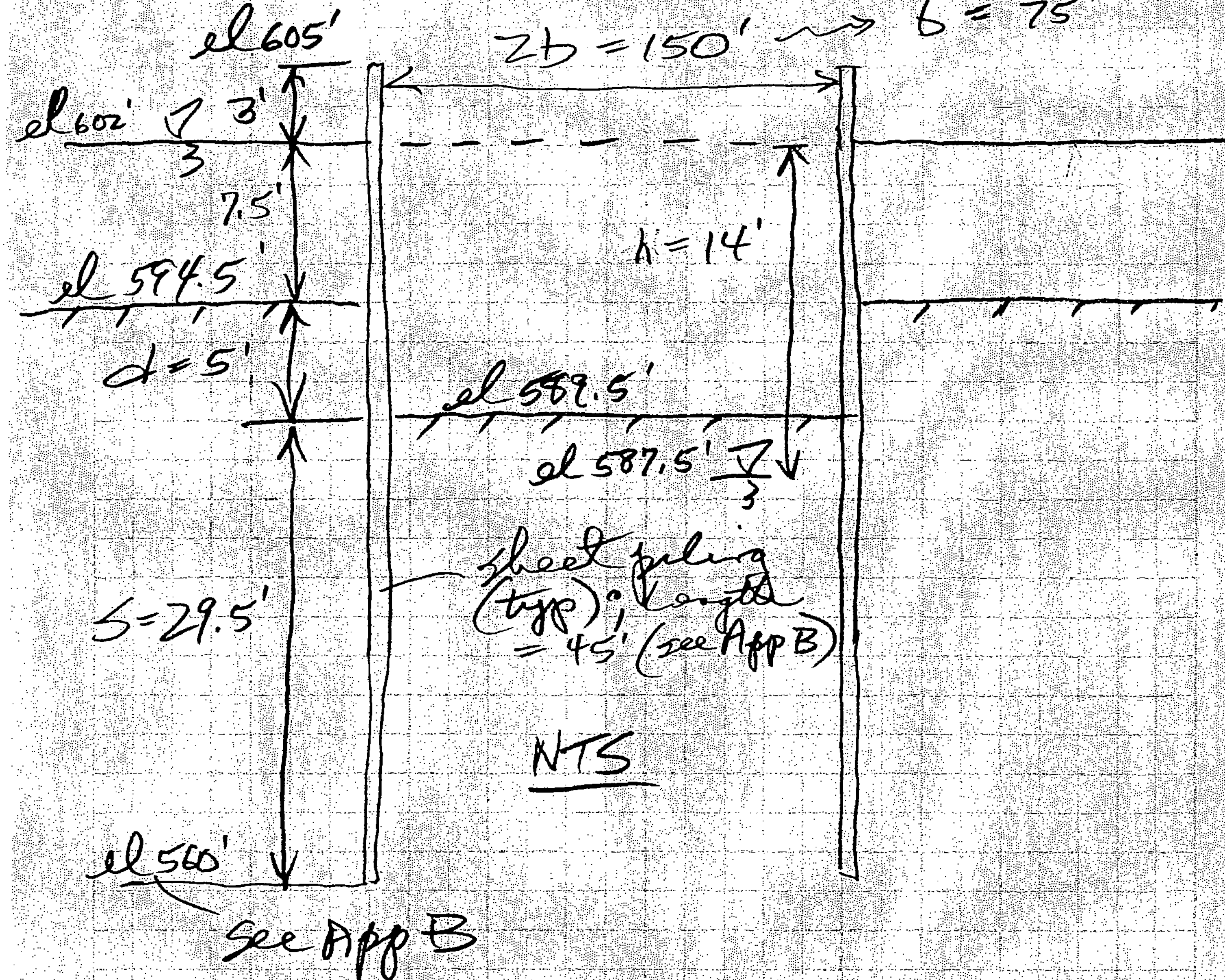

Based of Fig 5-33 of Harr (see pg 14)

$s/b = 29.5'/75' = .393$

$d/b = 5'/75' = .067$

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& $\therefore m = .44\ (\pm) \rightsquigarrow$ Fig 5-33 (Top)

then from Fig 5-36 (Bottom) → see pg 13

$$\frac{(I_E)S}{h} = .38$$

$$\downarrow$$

$$I_E = I_{E_{max}} = \frac{.38\,h}{S} = \frac{.38\,(14.5')}{29.5'}$$

$$= .187$$

$$\& \quad I_{E_{max}}(\text{TOTAL}) = .187 + \underbrace{(1.7)\times 10^{-5}}_{\approx 0}$$

$$= .187\ \text{ft/ft}$$

this value can then be compared to the critical exit gradient (I_{cr}) at which "piping" could occur to define an FS against piping as follows as noted on pgs 18 + 19:

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(14)

called sand boiling. Thus,

$$\sigma' = 0 = z\gamma' - \frac{hz}{H_2}\gamma_w \tag{5.21}$$

$$\frac{h}{H_2} = \frac{\gamma'}{\gamma_w} \tag{5.22}$$

The hydraulic gradient when the effective stress equals 0 is called the critical hydraulic gradient, i_{cr}, which can be expressed as follows

$$i_{cr} = \frac{(h/H_2)z}{z} = \frac{h}{H_2} = \frac{\gamma'}{\gamma_w} \tag{5.23}$$

Besides, according to the phase relationship of soil, the submerged unit weight is

$$\gamma' = \left(\frac{G_s - 1}{1 + e}\right)\gamma_w \tag{5.24}$$

where G_s is the specific gravity and e is the void ratio. The critical hydraulic gradient is then

$$i_{cr} = \frac{G_s - 1}{1 + e} \tag{5.25}$$

Since the G_s-value of sand is about 2.65 and its e-value is between 0.57 and 0.95, the critical hydraulic gradient for most sands is close to 1.0 according to the above equation.

Figure 5.33 shows watertight sheet piles. When the exit gradient (point A in the figure) is close to the critical hydraulic gradient, sand boiling occurs. Harza (1935) defines the factor of safety against sand boiling as follows

$$F_s = \frac{i_{cr}}{i_{max(exit)}} = \frac{I_{cR}}{I_{Emax}\ (\text{TOTAL})} \tag{5.26}$$

where $i_{max(exit)}$ is the maximum hydraulic gradient at the exit of seepage, which can be obtained with the flow net method.

Terzaghi (1922) found, according to many model tests with single rows of sheet piles, that the phenomenon of piping occurs within a distance of about $H_p/2$ from the sheet piles (H_p refers to the penetration depth of the sheet piles). Thus, to analyze the stability of single rows of sheet piles, we can take the soil column $H_p \times H_p/2$ in front of the sheet pile as an analytic object, as shown in Figure 5.33. The uplift force on the soil column would be

$$U = (\text{the volume of the soil column}) \times (i_{av}\gamma_w) = \tfrac{1}{2}H_p^2 i_{avg}\gamma_w \tag{5.27}$$

where i_{avg} is the average hydraulic gradient of the soil column. The downward force of the soil column (i.e. the submerged weight) is

$$W' = \frac{1}{2}H_p^2(\gamma_{sat} - \gamma_w) = \frac{1}{2}H_p^2\gamma' \tag{5.28}$$

Ref: "Deep Excavation Theory & Practice"; C.Y. Ou

19

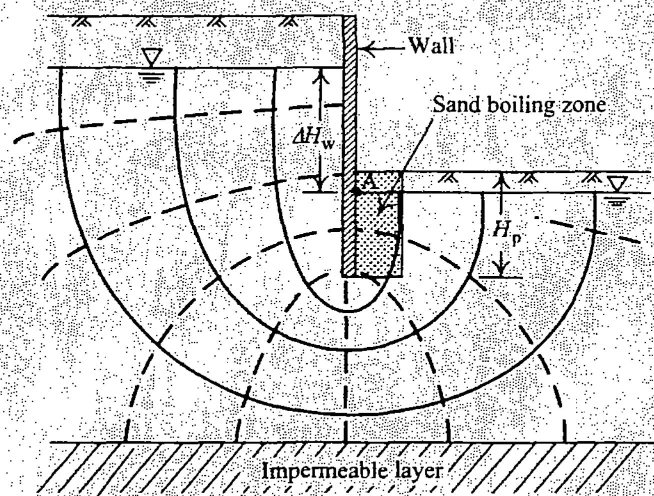

Figure 5.33 Seepage in soil below sheet piles.

Therefore, the factor of safety is

$$F_s = \frac{W'}{U} = \frac{\gamma'}{i_{avg}\gamma_w} \tag{5.29}$$

According to Eq. 5.29, provided the computed factor of safety is too small, we can consider placing filters at the exits of seepage. Assuming the weight of the filters is Q, the factor of safety will be

$$F_s = \frac{W' + Q}{U} \tag{5.30}$$

In general, the required F_s for the above equation should be greater than or equal to 1.5 (JSA, 1988; TGS, 2001).

As this equation shows, if sand boiling or piping occurs, in addition to evacuating the workers and equipment as soon as possible, the possible remedial measures include dewatering to reduce the water pressure and dumping permeable soils onto the excavation surface to increase the value of the numerator in the equation.

Marsland (1953) conducted a series of model tests to explore the phenomenon of piping in excavations in sand and obtained the results, which were later adopted by NAVFAC DM7.1 (1982), as shown in Figure 5.34. Figures 5.34a and 5.34b show the results with the impermeable layers located infinitely deeply and with the impermeable layer within a finite depth, respectively. NAVFAC DM 7.1 suggested that the reasonable factor of safety against piping in an excavation be around 1.5–2.0. We can see from the figure that as long as the allowable factor of safety (F_s), the excavation depth (H_e), and the distance of the excavation surface to the impermeable layer (D) are known, we can obtain the required penetration depth of the retaining wall (H_p) against piping.

WESTON SOLUTIONS

SHEET 20 of

CLIENT/SUBJECT W.O. NO.

TASK DESCRIPTION TASK NO.

PREPARED BY DEPT DATE APPROVED BY

MATH CHECK BY DEPT DATE

METHOD REV. BY DEPT DATE DEPT DATE

$$FS = \frac{I_{CR}}{I_{E\,max}\text{ (TOTAL)}} > 4 \rightarrow 5$$

Ref: Harr, pg 26

& $I_{CR} = \frac{G_s - 1}{1+e}$ → see pg 18 eqn 5.25

G_s = specific gravity of SP/SM excav. bottom soils

- assume $G_s = 2.65$ (typ range for granular soils is 2.62 → 2.68)

e = insitu void ratio of SP/SM excav bottom soils

- can estimate from

$Se = wG_s$

S = degree of saturation of SP/SM soils

- assume 100% (fully saturated) ∴ $S = 1.0$

WESTON SOLUTIONS

SHEET 21 of ____

CLIENT/SUBJECT ____ W.O. NO. ____

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DEPT ____ DATE ____

w = in situ water content of soils

- from measured lab data (see Attachment) by SEH.

- 10 data pts available

- use ave value for "w"

$$w = w(\text{ave}) = \frac{24+22+18+21+21+24+22+27+25+21}{10}$$

$$= 22.5\% = .225$$

then $e = \dfrac{w G_s}{S} = \dfrac{(.225)(2.65)}{(1.0)}$

$$= .596$$

& $I_{CR} = \dfrac{G_s - 1}{1+e} = \dfrac{2.65-1}{1+.596} = 1.034$

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SHEET 22 of

CLIENT/SUBJECT W.O. NO.

TASK DESCRIPTION TASK NO.

PREPARED BY DEPT DATE

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Note: I_{CR} is typ. near 1.0 for most granular soils unless these are either in a very dense or very loose relative density condition.

$$FS = \frac{I_{CR}}{I_{E_{max}}(\text{TOTAL})} = \frac{1.034}{.187}$$

$$= 5.53$$

$> 4 \to 5$

~~as suggested~~ by ~~Harr~~

∴ OK

WESTON SOLUTIONS

SHEET ____ of ____

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TASK DESCRIPTION ____________________ TASK NO. __________

PREPARED BY __________ DEPT ______ DATE ______

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DEPT ______ DATE ______

ATTACHMENT

A-1

SEH

MEMORANDUM

□ ST. PAUL, MN □ MINNEAPOLIS, MN □ ST. CLOUD, MN ■ CHIPPEWA FALLS, WI □ MADISON, WI □ GRIFFITH, IN

TO: John Guhl

FROM: Glenn Bruxvoort GRB

DATE: May 8, 1996

RE: Ashland Lakefront Property Soils Results

Enclosed are the results of the soils lab testing performed on samples from your Ashland Lakefront Property Project (Project No. WIDNR9401.01). The samples were analyzed in general accordance with ASTM D422, D854, D4318, D4959, D5084, D2974, and D1558 standards.

Unless you request differently, the samples will be discarded in 30 days in accordance with our standard policy. If you require additional information or have further questions please call me.

CAU/cau/JJT

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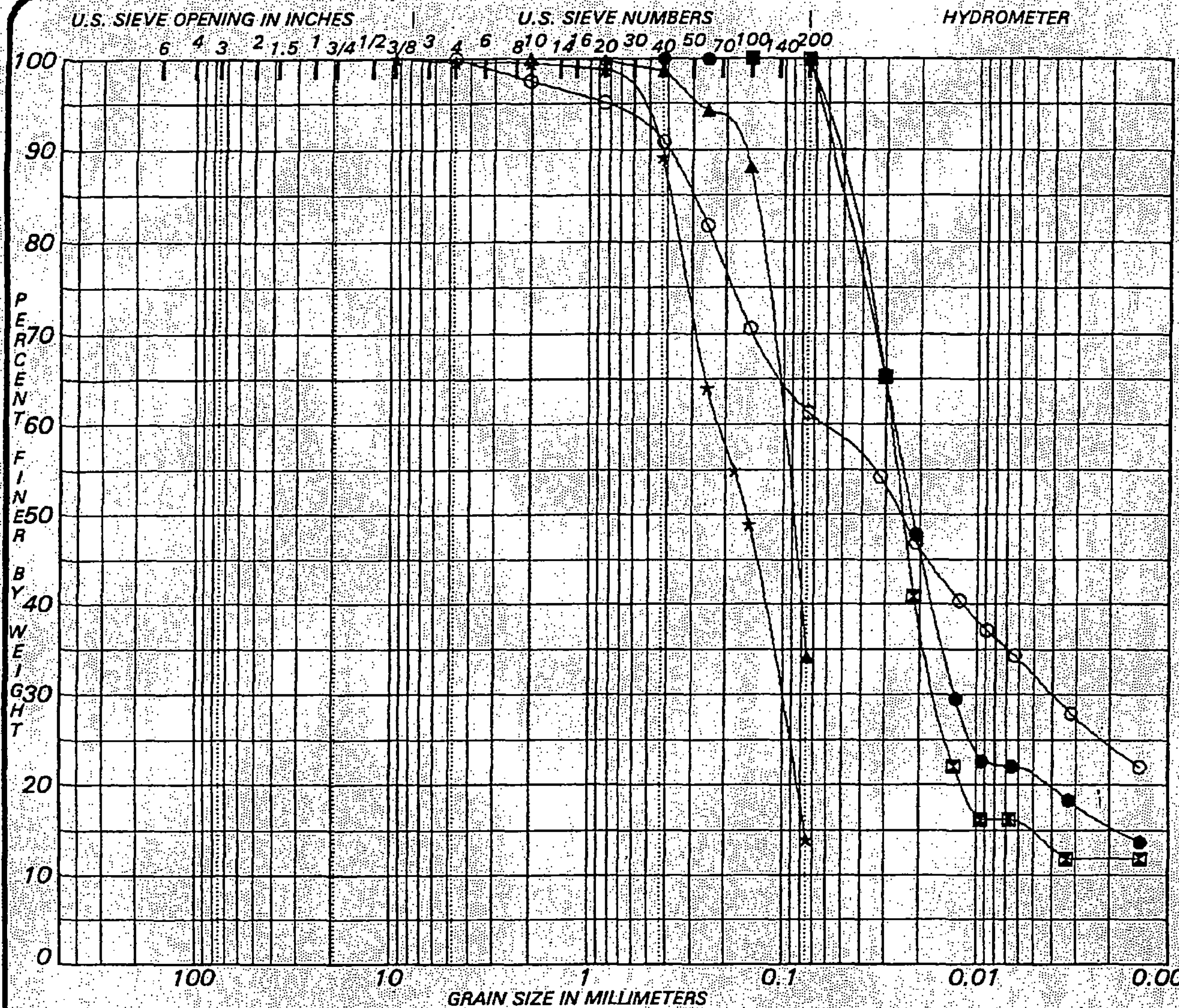

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification	Classification	MC%	LL	PL	PI	Cc	Cu
●	2400N 1200E (20-24')	SILT,ML	21	NP	NP	NP		
⊠	2400N 1200E (8-10')	SILT,ML	21	NP	NP	NP		
▲	2400N 1200E (4-6')	SILTY SAND,SM	24	NP	NP	NP		
★	2500N 2100E (4-6')	SILTY SAND,SM	22	NP	NP	NP		
⊙	2700N 2100E (6-8')	SANDY LEAN CLAY,CL	18	28	14	15		

	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	2400N 1200E (20-24')	0.43	0.03	0.013		0.0	0.1	84.3	15.6
⊠	2400N 1200E (8-10')	0.15	0.03	0.016		0.0	0.1	88.1	11.8
▲	2400N 1200E (4-6')	2.00	0.10			0.0	65.9	34.1	
★	2500N 2100E (4-6')	9.50	0.22	0.103		0.4	85.7	13.9	
⊙	2700N 2100E (6-8')	4.75	0.06	0.004		0.0	38.7	36.8	24.5

PROJECT Ashland Lakefront Property - Ashland, WI

JOB NO. WIDNR9401.01
DATE 5/2/96

GRADATION CURVES

SEH
Chippewa Falls

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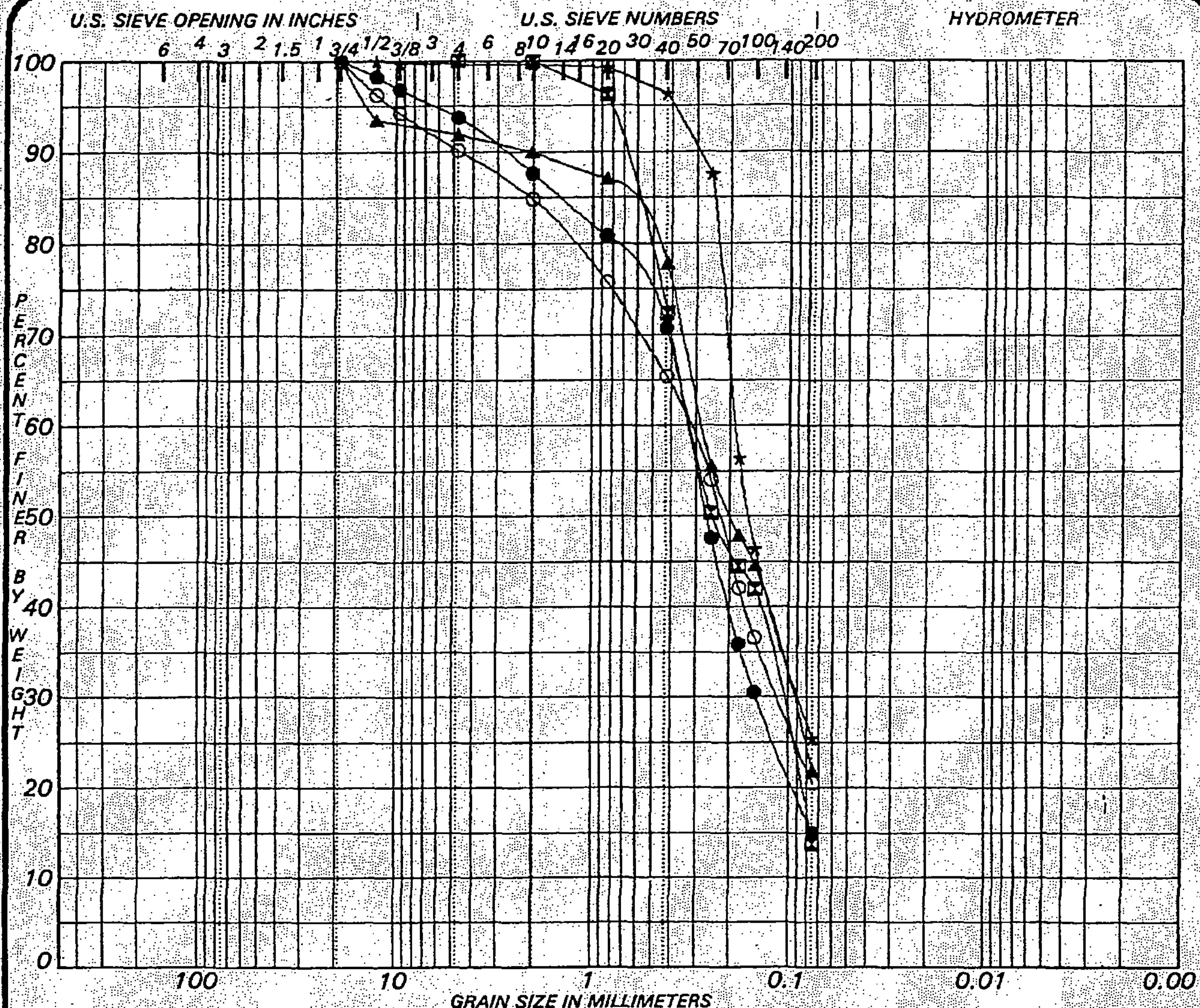

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification	Classification	MC%	LL	PL	PI	Cc	Cu
●	2700N 2100E (4-6')	SILTY SAND,SM	18	NP	NP	NP		
⊠	2600N 1700E (0-4')	SILTY SAND,SM	21	NP	NP	NP		
▲	2600N 1200E (4-6')	SILTY SAND,SM	21	NP	NP	NP		
★	2900N 2000E (6-8')	SILTY SAND,SM	24	NP	NP	NP		
⊙	2500N 2600E (8-10')	SILTY SAND,SM	22	NP	NP	NP		

	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	2700N 2100E (4-6')	19.00	0.33	0.147		6.3	78.7	15.0	
⊠	2600N 1700E (0-4')	4.75	0.32	0.112		0.0	86.2	13.8	
▲	2600N 1200E (4-6')	19.00	0.28	0.096		8.2	69.9	21.9	
★	2900N 2000E (6-8')	12.50	0.19	0.087		0.3	74.3	25.4	
⊙	2500N 2600E (8-10')	19.00	0.33	0.113		9.9	69.6	20.5	

PROJECT Ashland Lakefront Property - Ashland, WI

JOB NO. WIDNR9401.01
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GRADATION CURVES

SEH
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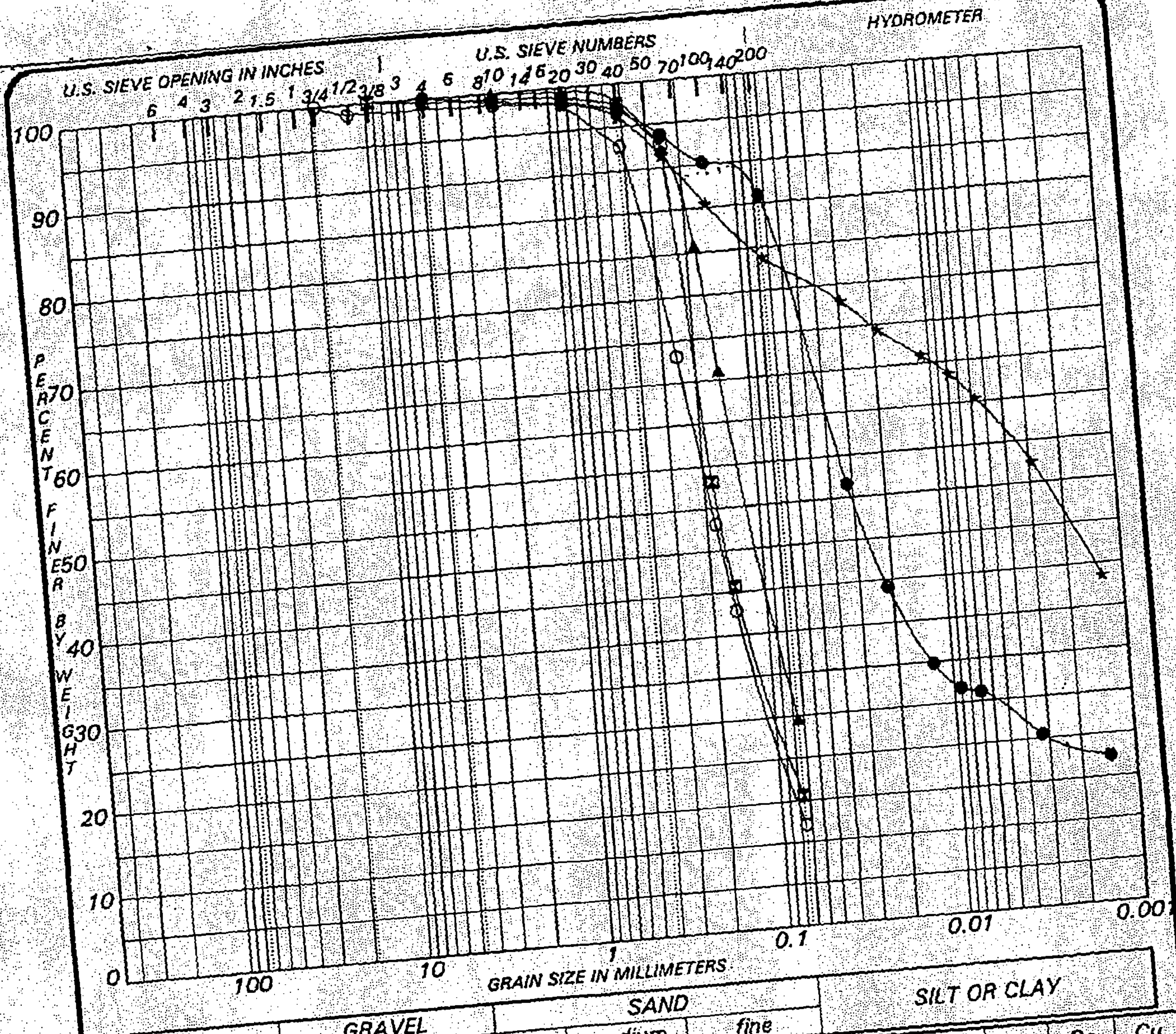

	Specimen Identification	Classification	MC%	LL	PL	PI	Cc	Cu
●	2500N 2600E (10-12')	SILT,ML	19	NP	NP	NP		
⌧	3100N 1300E (0-4')	SILTY SAND,SM	27	NP	NP	NP		
▲	3100N 2300E (6-8')	SILTY SAND,SM	25	NP	NP	NP		
★	3100N 2300E (12-14')	LEAN CLAY with SAND,CL	23	41	16	25		
○	2900N 1500E (4-6')	POORLY GRADED SAND with SILT,SP-SM	21	NP	NP	NP	0.96	3.1

	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	2500N 2600E (10-12')	4.75	0.04	0.013		0.0	14.2	67.1	18.7
⌧	3100N 1300E (0-4')	9.50	0.19	0.112		0.5	84.0	15.5	
▲	3100N 2300E (6-8')	4.75	0.14	0.082		0.0	75.4	24.6	
★	3100N 2300E (12-14')	4.75	0.01			0.0	21.3	32.7	46.0
○	2900N 1500E (4-6')	19.00	0.22	0.122		1.3	86.6	12.1	

PROJECT Ashland Lakefront Property - Ashland, WI

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